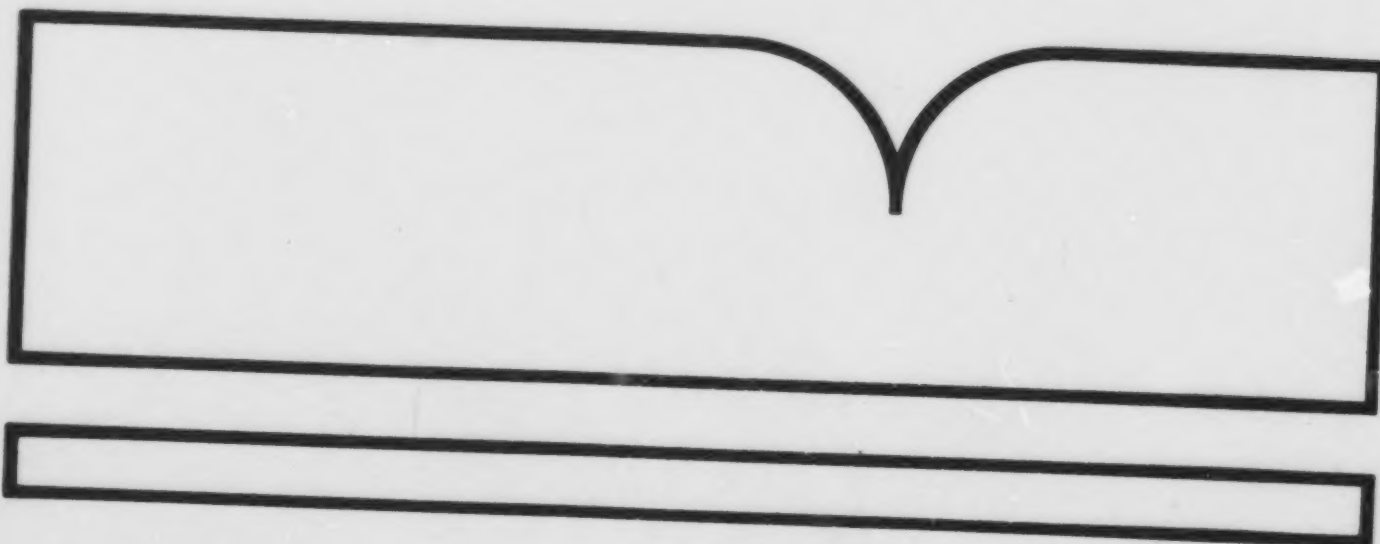


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April 12-13, 1984

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PROCEEDINGS
OF
STORMWATER AND WATER QUALITY MODEL
USERS GROUP MEETING
April 12-13, 1984

Edited by

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ENVIRONMENTAL RESEARCH LABORATORY
OFFICE OF RESEARCH AND DEVELOPMENT
U.S. ENVIRONMENTAL PROTECTION AGENCY
ATHENS, GA 30613

DISCLAIMER

The work described in these papers was not funded by the U.S. Environmental Protection Agency. The contents do not necessarily reflect the views of the Agency and no official endorsement should be inferred.

FOREWORD

A major function of research and development programs is to effectively and expeditiously transfer technology developed by those programs to the user community. A corollary function is to provide for the continuing exchange of information and ideas between researchers and users, and among the users themselves. The Stormwater and Water Quality Model Users Group, sponsored jointly by the U.S. Environmental Protection Agency and Environment Canada/Ontario Ministry of the Environment, was established to provide such a forum. The group has recently widened its interest to include models other than the Stormwater Management Model and other aspects of modeling water quality in urban and natural waters. This report, a compendium of papers presented at the April 1984 Users Group meeting in Detroit MI, is published in the interest of disseminating to a wide audience the work of group members.

Rosemarie C. Russo, Ph.D.
Director
Environmental Research Laboratory
Athens, Georgia

ABSTRACT

This proceedings includes 17 papers on topics related to the development and application of computer based mathematical models for water quantity and quality management. The papers were presented at the semi-annual meeting of the Joint U.S.-Canadian Stormwater and Water Quality Model Users Group held on April 12-13, 1984, in Detroit, Michigan.

Two papers discuss the application of microcomputers to real-time control of combined sewer overflows and to the estimation of nutrient and pollutant loadings. In separate papers, the SWMM program is applied to hydraulic modeling in an unsteady pressure flow regime and to sensitivity analysis of water quality predictions. Other model applications include HSPF to simulate stormwater and water quality aspects of ponds and WASP to simulate acidification of lakes. A combined hydrologic time series and topographic database manager (CHGTSM) is presented, and revisions to the QUAL-2 water quality model are discussed.

Aspects of rainfall modeling are examined in a program for analyzing rainfall inputs in computing storm dynamics and in a system for acquiring and processing rain data using Apple II work-alikes. Estuary studies include developing flow and loading inputs for steady state models and modeling negatively buoyant thermal discharge.

Milwaukee's modeling approach to water quality and sewer system analysis is described, and Ohio's method for determining the reaeration coefficient for streams is presented. Additional papers examine kinematic distribution of detention storage, discuss the need for hydrologic model validation, and provide water temperature modeling guidance.

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The Stormwater and Water Quality Model Users Group relies on local hosts to make arrangements for meeting rooms and participant housing. The hosts for the meeting reported in this proceedings were Dr. Ralph H. Kummier of the Department of Chemical and Metallurgical Engineering, Wayne State University; Dr. Thomas Heidtke of the Department of Civil Engineering, Wayne State University; and James A. Anderson of Urban Science Applications, Inc.

MILWAUKEE'S INTEGRATED MODELING APPROACH
TO WATER QUALITY AND SEWER SYSTEM ANALYSIS

by

Nancy U. Schultz, CH2M HILL, Milwaukee
and

Gary A. Gagnon, Milwaukee Metropolitan Sewerage District

ABSTRACT

Milwaukee's Water Pollution Abatement Program is a multi-year, multi-agency program geared to plan, design, and construct sewage collection and treatment facilities required for the area's surface waters to meet water quality criteria. The efforts are coordinated through a program management office which conducts systemwide studies and coordinates site specific projects. Over the past years the program management office has developed and/or applied numerous computer models to aid in this analysis and in coordination of the project. This paper describes how the several models and their respective data bases have been integrated into a modeling approach which emphasizes strengths of the various individual models and acknowledges the weaknesses by supplementing with other models.

The surface water models used in the program to date include STORM, SWMM, SAM, CAM, HSP, several data base and statistical packages, and some single purpose models written specifically for the Milwaukee program. The paper presents a chronology of the use of these models, discussing why a specific model was chosen for a given application, how it was used, key results and limitations of its application. Uses discussed in the paper include:

- o Water quality impact analysis
- o Sewer system relief needs evaluation
- o Infiltration/inflow prediction
- o Storage volume calculation
- o Combined sewer design flow calculation
- o Hydraulic analyses
- o Instrumentation and control needs definition

The paper concludes with a description of anticipated future uses of the integrated modeling approach of the Milwaukee Pollution Abatement Program.

INTRODUCTION

The Milwaukee Water Pollution Abatement Program is a joint effort of the Milwaukee Metropolitan Sewer District (MMSD), CH2M HILL and numerous other consultants to plan, design, and construct all sewerage facilities necessary to meet federal, state, and court ordered pollution control regulations. The program planning area covers 424 square miles, drained by over 12 million linear feet of sewers and interceptors. Twenty seven square miles are drained by combined sewers. Milwaukee's combined sewer area connects to the separate sewer system only at the regulators controlling flow to the interceptors. Unlike most combined systems, upstream separate sewers do not flow into combined sewers.

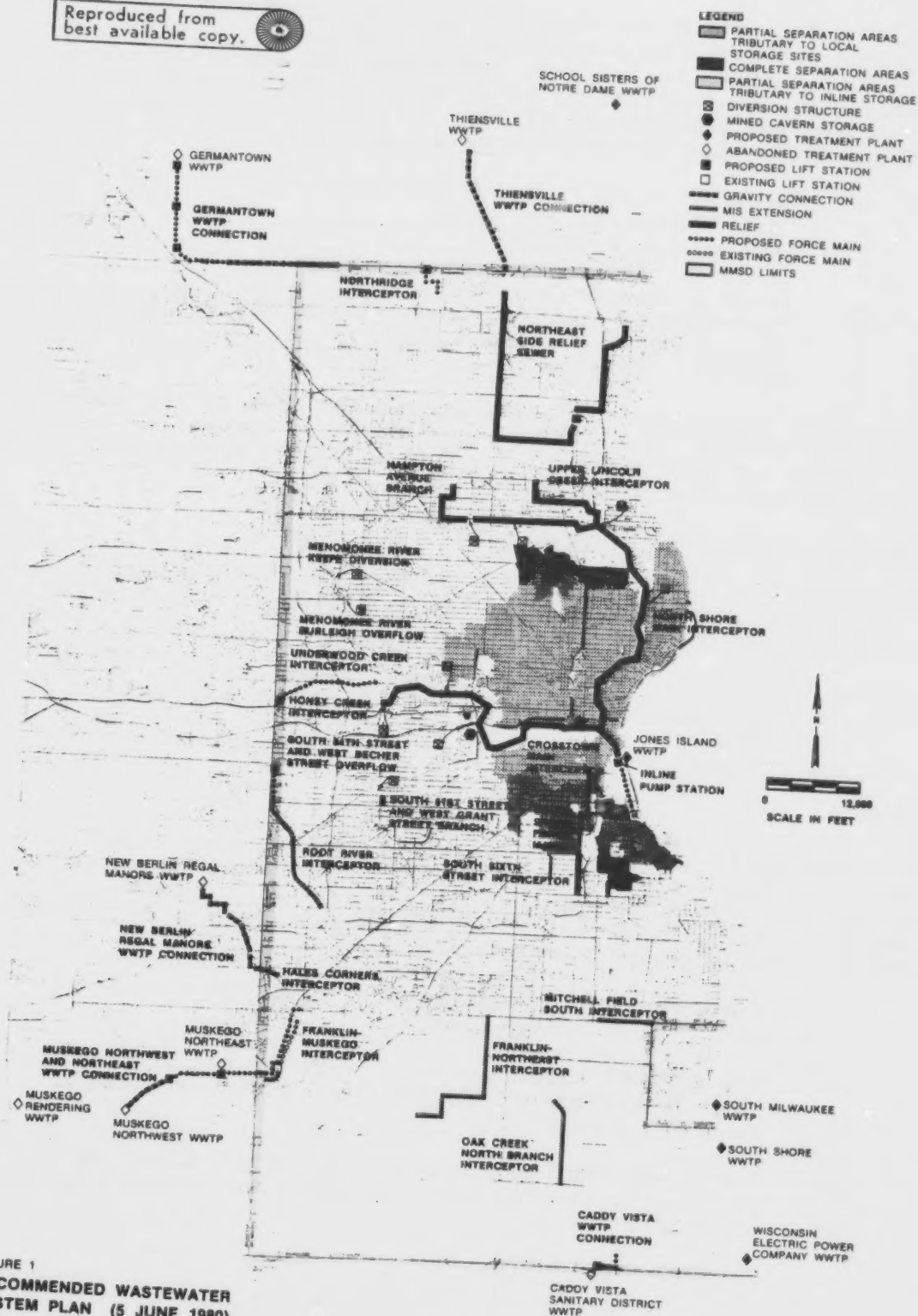
Originally, all sewers flowed toward the center city Jones Island wastewater treatment plant which is shown in Figure 1. Since the late 1960's, many sewer diversions have been installed to allow discretionary routing of about two-thirds of the separate sewer system towards the new South Shore wastewater treatment plant.

When the program started, separate sewers bypassed to the rivers several times per year, and combined sewers overflowed during most rainstorms. Regulations require that essentially all separate sewer bypasses be eliminated by 1986. The regulatory agencies have required that even the storm of record cannot cause separate sewer bypasses from the future, upgraded system. In addition, combined sewer overflows must be controlled. Current estimates require a half year level of protection for the combined sewers, but ongoing studies will determine the final requirements.

Several years of facility planning have concluded the control objectives can be realized through:

- o Infiltration/inflow reduction
- o Sewer relief construction
- o Treatment plant rehabilitation
- o Construction of a tunneled interceptor to be used for conveyance and storage of both separate sewage and combined sewer overflows
- o Construction of connecting sewers and control works to enable joint, but selective, use of the conveyance/storage system

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Planning facilities for such a large area in a short time required extensive data handling and computation. Computers have been used extensively to facilitate the computations and enable detailed analysis of alternative pollution control facilities and operational plans. This paper discusses computerized water resources models used for:

- o Combined Sewer Overflow analysis
- o Infiltration/Inflow Study
- o System Wide Analysis
- o Environmental Assessment
- o Sewer System Evaluation Survey
- o Predesign of Combined Sewer Collectors
- o Operational Control Analysis
- o Hydraulic Design

This paper summarizes the applications and uses of the water resources models. The bibliography contains several references for more thorough discussion of the models and their individual applications.

COMBINED SEWER OVERFLOW ANALYSIS

An analysis of combined sewer overflows and their control requirements was undertaken in the late 1970's. Three water resources models were used in these studies:

- o Stormwater Management Model (SWMM)
- o Storage, Treatment, Overflow, and Runoff Model (STORM)
- o Harper/Owes Receiving Water Quality Model

SWMM

The SWMM model was developed in the late 1960's by Water Resources Engineers and is currently distributed and supported by the EPA Center for Water Quality Modeling in Athens, Georgia. Three modules of the SWMM package were used for the combined sewer study in Milwaukee:

- o The RUNOFF block was used to calculate stormwater runoff from design storms.
- o The QUALITY block was used to calculate quality characteristics of the combined stormwater and sanitary sewage.
- o The TRANSPORT block was used to route flows and pollutants through the combined sewer network to the outfalls.

The primary outputs of the SWMM application were calculated pollutographs used as input to the receiving water quality analysis and peak flows used for preliminary sizing of collector sewers proposed to carry flows from existing outfall locations to future storage locations.

This early SWMM application was limited by sparse data availability, hence sewer networks were only roughly approximated and the hydrology portion of the analysis was not calibrated. Furthermore, the SWMM TRANSPORT assumption that flows exceeding sewer capacity store at upstream locations significantly altered the shape of outflow hydrographs and underestimated peak flows. Peak flows calculated by SWMM were increased 20 percent in surcharged systems to compensate for the assumed storage.

STORM

The STORM is distributed and supported by U.S. Army Corps of Engineers Hydrologic Engineering Center in Davis, California. The continuous hydrology portion of the model was used to calculate total combined sewer overflow in the Milwaukee area for summer periods from 1941 through 1974. The resultant flows were analyzed to evaluate storage volumes required to achieve target levels of protection against combined sewer overflow.

Like the SWMM analysis, the STORM analysis was not calibrated due to a lack of available hydrologic data. Furthermore, the analysis failed to consider the early spring snowmelt periods which significantly impact runoff volumes in the upper midwest.

HARPER/OWES RECEIVING WATER QUALITY MODEL

Messrs. Harper and Owes developed a water quality model specifically adapted for evaluating the water quality impact of combined sewer overflows into the lower Milwaukee River. The model, coupled with overflow pollutographs from SWMM, was used to evaluate receiving water quality response to several storms and several control alternatives.

Calibration of the Harper/Owes model was limited by: the need to represent upstream loads by averages since storm specific data were lacking; a lack of data for hydrologic calibration; and an inability to simulate the observed immediate response to combined sewer overflow events. The model was adjusted to simulate bottom sediment disturbance as an immediate dissolved oxygen sink when subsurface discharge of combined sewage occurred. This modification improved calibration and indicated a need for further research regarding the oxygen demand of disturbed sediments.

COMBINED SEWER OVERFLOW ANALYSIS RESULTS

The model analysis were coupled with economic, engineering, and environmental evaluations to conclude:

1. A half year level of protection would meet receiving water quality goals.
2. Partial separation of sewers (providing storm sewers but not disturbing private property to disconnect building drains) represented the most cost effective method of achieving water quality goals, particularly in view of the then pending court case which would have required elimination of all combined sewer overflows. Later developments in the court case resulted in the partial separation recommendation being dropped from the pollution abatement program.

INFILTRATION/INFLOW STUDY

An infiltration/inflow (I/I) study of the separate sewer system was conducted coincident with, but independent of, the Milwaukee combined sewer overflow study. Two water resources models were used in the I/I analysis:

- o Milwaukee I/I flow model
- o System Analysis Model (SAM)

MILWAUKEE I/I FLOW MODEL

The Milwaukee I/I flow model was developed from pre-existing separate sewer flow data. Flow data were available for several locations throughout the system for a short period (two to three years) and for a longer period (about 10 years) at the South Shore treatment plant. The short records were used to develop characteristic hydrograph shapes for various regions of the service area. The hydrographs were scaled to represent a storm of record throughout the system on the basis of the peak to peak ratio comparing local, short term records with the system, long term record at the treatment plant. Attempts to relate I/I peak flows to rainfall and sewer basin characteristics were unsuccessful since no significant correlation could be demonstrated between the rainfall and sewer flow data.

SAM

SAM was developed by CH2M HILL in the late 1960's to aid municipalities in evaluating storm and sanitary sewer systems. The I/I studies used the kinematic wave sewer hydraulics

approach to route the I/I hydrographs discussed previously through the Milwaukee Metropolitan Intercepting System (MIS). The analysis calculated routed peak flows in all sections of the MIS and identified sewer reaches requiring relief to accommodate existing and future flows. SAM also calculated hydraulic gradeline elevations required to pass peak flows through surcharged sewers. Relief was deemed necessary only where the hydraulic gradeline exceeded flag elevations above which property damage could be expected.

I/I STUDY RESULTS

The I/I analyses concluded that a 48 percent reduction of the infiltration/inflow could be cost effectively achieved in the Milwaukee separate sewer system. Even with that I/I reduction, relief sewers would be necessary in several locations. The most significant relief needs required large diameter interceptors paralleling the Milwaukee and Menomonee Rivers. Analysis indicated the most cost effective alternative would involve deep rock oversized tunneled sewers with capacity for inline storage and conveyance of excess peak flows with eventual pumpout to treatment. Later studies reduced the percentage of I/I which could be cost effectively removed from the system, necessitating additional relief sewer projects.

SYSTEM WIDE ANALYSIS

A system wide analysis was undertaken to evaluate the potential for joint use of the tunneled inline storage/conveyance system for both separate sewer and combined sewer overflow relief. Two water resources models were developed for this analysis:

- o Regression Flow Model
- o Storage Analysis Model

REGRESSION FLOW MODEL

A sewer flow model which consistently projected both separate sewer flows and combined sewer flows for a wide variety of events was needed to evaluate the joint use of one storage/conveyance facility for both systems. Hydrologic theory indicated both I/I and combined sewer flow should respond (albeit to differing degrees) to the hydrologic processes governed by land use, precipitation, soil moisture, snow melt, evapotranspiration, etc.

The Southeastern Wisconsin Regional Planning Commission (SEWRPC) had recently completed a thorough analysis of available hydrologic data through the calibration of the Hydrocomp Simulation Program (HSP) version of the Stanford Watershed Model. That analysis output a 39.6 year record

(January 1940 through July 1979) of hourly calculations of subsurface flows, impervious surface runoff and pervious surface runoff in units of inches per unit area.

Available sanitary sewer flow data, sewer basin data, and the continuous hydrologic output from HSP were tested in a variety of linear and psuedo linear regression models. The following relationships were found to adequately predict separate sewer infiltration and inflow:

$$\begin{aligned}\text{Infiltration} &= D + E * (\text{SUBRO}) \\ \text{Inflow} &= B * (\text{SUBRO}) + C * \frac{(\text{IMPRO} + \text{OLFRO})}{2}\end{aligned}$$

where:

$$\begin{aligned}\text{SUBRO} &= \text{subsurface flow per unit area} \\ \text{IMPRO} &= \text{impervious area runoff per unit area} \\ \text{OLFRO} &= \text{pervious area runoff per unit area} \\ B &= 0.486 * \text{number of buildings} \\ C &= [0.398 * \text{number of buildings} + 18.12 * \text{number} \\ &\quad \text{of commercial buildings}] * 10^{-3} \\ D &= 0.845 * \text{feet of sewer} + 1062 * \text{number of} \\ &\quad \text{industrial and commercial buildings} \\ E &= 1.001 * \text{feet of laterals} * 10^{-4}\end{aligned}$$

Combined sewer flow could be similarly predicted from the same hydrologic data base using the relationship:

$$\text{Stormwater flow} = \text{infiltration} + \text{IMP} * \text{IMPRO} + (1 - \text{IMP}) * \text{OLFRO}] * \text{area drained}$$

where

infiltration, IMPRO and OLFRO are as defined above
IMP = impervious area fraction

STORAGE ANALYSIS MODEL

A simplified storage analysis model was developed to route separate and combined sewer flows through the system to either treatment, storage or river overflows under a variety of system configurations. The model employed linear storage routing concepts to route the entire 39.6 year record of flows developed with the regression flow model. A daily time step was used to speed evaluation of the more than 50 alternative configurations studied. Variables addressed in the alternatives included:

- o Inline storage volume
- o Offline storage volume
- o Available treatment capacity
- o Pumpout rate

SEWER SYSTEM EVALUATION SURVEY

The several models developed in the I/I analysis and the System Wide Analysis (SAM, Regression, Storage) were used jointly in the sewer system evaluation survey to determine conveyance and storage facility requirements to offset uncontrolled infiltration and inflow. These models, along with computerized sewer inspection records, were used to conclude that approximately 13 percent of existing I/I could be cost effectively removed from the system and to define sewer relief needs.

ENVIRONMENTAL ASSESSMENT

Water resources computer models were used to evaluate receiving water impacts of the Milwaukee Water Pollution Abatement Program. Three models were used in early phases of the study:

- o Hydrocomp Simulation Program (HSP)
- o Milwaukee Harbor Circulation Model
- o Plume dispersion model

HSP

HSP was used to calculate nonpoint loads and receiving water response to the combined impact of point and nonpoint source loadings under a variety of decentralization and regionalization alternatives. Application of HSP was greatly simplified since extensive hydrometeorologic data base development and model calibration had previously been completed on a regional scale by the Southeastern Wisconsin Regional Planning Commission.

HSP was used to evaluate receiving water response only in riverine reaches unaffected by Lake Michigan backwater impacts.

MILWAUKEE HARBOR CIRCULATION MODEL

The Milwaukee Harbor Circulation Model was developed by University of Wisconsin-Milwaukee researchers. It is a hydrodynamic model which represents the wind and seiche driven currents characteristic of the Milwaukee outer harbor. It was used to characterize dominant circulation, pollutant transport, and dispersion of discharges from the Milwaukee River and the Jones Island wastewater treatment plant into the Milwaukee Harbor.

PLUME DISPERSION MODEL

A plume dispersion model was used to calculate pollutant dispersion from both the Jones Island and South Shore

discharges under a variety of dominant current and nearshore wind conditions. The analyses were used to project treated discharge impacts on nearshore waters, beaches, and water intakes with alternative discharge locations (nearshore and deep water).

COMBINED SEWER PREDESIGN

Predesign analysis of the combined sewer collector system used the System Analysis Model for three purposes:

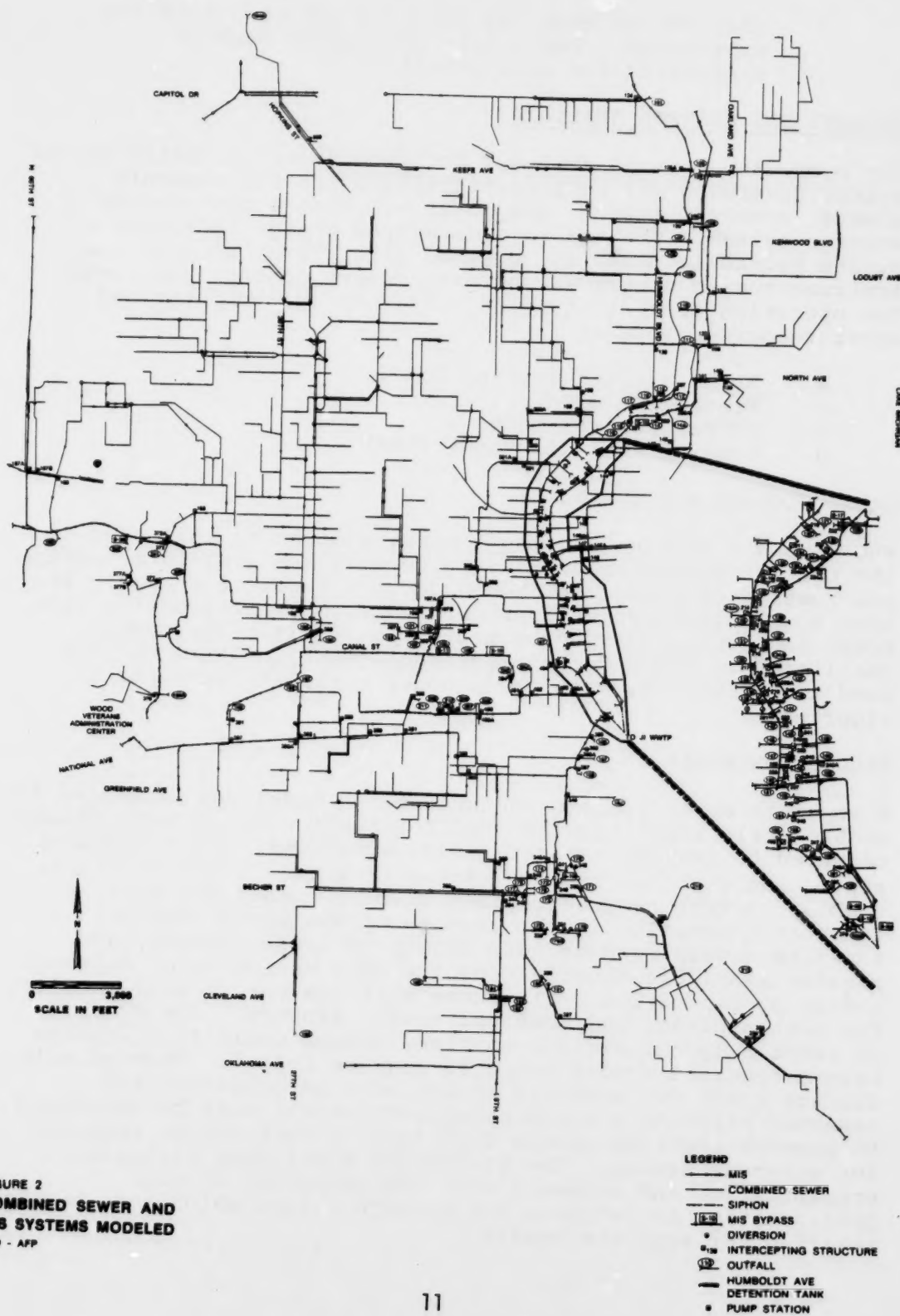
- o Calculation of storm runoff hydrographs
- o Routing and calculation of peak design flows
- o Evaluation of hydraulic interactions between combined sewers and interceptors

Milwaukee combined sewers greater than 36 inches in diameter and numerous diversions in the existing system were included in the hydraulic routing model (Figure 2). The area drained was divided into subbasins of 1 to 20 acres from which runoff hydrographs were calculated for the design storm and routed through the sewer systems tributary to each of the 114 outfall locations. Collector sewer models routed the outfall hydrographs to the proposed dropshafts connecting combined sewer overflows to the tunnel conveyance/storage system. The collector sewer models were used for initial pipe sizing and hydraulic profile calculations.

SAM was also used in an iterative mode to balance flows from the combined sewers through the intercepting structures to the intercepting sewers. Since the intercepting sewers surcharge during storms, few intercepting structures have free outfalls. Flows through the intercepting structures are governed by hydraulic heads both in the interceptor and in the combined sewers. SAM was used to calculate interceptor hydraulic gradelines under a variety of intercepting structure flow assumptions.

SAM was chosen for the combined sewer collector predesign even though SWMM had previously been used in the combined sewer area for three reasons:

1. SWMM had been applied at a large scale, planning level. Considerable detail had to be added to the SWMM representation if it was to be used.
2. SWMM had no simple means for addressing surcharged sewers. Many of the combined sewers surcharge under the design storm and SAM directly addresses the surcharge flow.
3. The analysis had to be completed in six to eight months using eight people from five different firms,



only one of whom had previous computer modeling experience. SAM's data management package simplified the user interface.

OPERATIONAL CONTROL ANALYSIS

The operation control analysis involved detailed definition of system diversion and control facilities for the separate sewers, combined sewers, and joint use tunnel conveyance/storage system. The analysis developed design hydrographs, system hydraulic profiles, control algorithms, process and instrumentation diagrams, and preliminary control setpoints. The operation control analysis utilized four computerized water resources models:

- o MACRO storage routing model
- o Prediction model
- o TUNNEL control model
- o Control Analysis Model (CAM)

MACRO STORAGE ROUTING MODEL

MACRO uses a storage routing concept similar to that used in the earlier System Wide Analysis to evaluate long-term storage and routing of both combined sewage and separate sewage. It uses hourly flow projections derived from the regression flow model and routes them through capacity limited conveyance facilities to the conveyance/storage tunnel. The model was used to evaluate storage needs under a variety of control algorithms.

PREDICTION MODEL

A separate sewer I/I storage prediction model was developed to enable early identification and reservation of storage volumes required to prevent separate sewer overflows. Both separate sewers and combined sewers respond to snowmelt and rainfall. Combined sewers respond rapidly to many events and will frequently overflow in the absence of available storage. Separate sewers respond more slowly to larger events, and require available storage capacity only during major snowmelt and/or precipitation. Milwaukee will use one storage facility for both separate and combined sewage storage. The dynamics of sewer response dictate combined sewage could fill storage before separate sewers begin to require relief. Consequently, storage space for separate sewage must be predicted and reserved prior to a storm event and control must be exercised to prevent combined sewage from filling that volume reserved for separate sewage. The prediction model uses historic precipitation and snowmelt data and projected 24 hour precipitation to estimate the storage volume which must be reserved for separate sewage.

The prediction model and MACRO were used jointly to develop and test a control strategy which allows combined sewage to fill only that storage volume not required for separate sewage, then allows combined sewage spills to the rivers while reserving the remaining storage volume for separate sewage. Using a 1.5 safety factor, it is predicted that this control strategy will result in no separate sewage spills to the river and about two combined sewage overflows per year. Without the prediction model, storage volumes would have to be nearly doubled to achieve the same levels of protection.

TUNNEL CONTROL MODEL

The TUNNEL control model performs detailed calculations of how the control strategy for controlling inflows to the storage/conveyance tunnel will alter the hydrograph entering the tunnel. It was used to generate critical hydrographs to be used for tunnel hydraulic design.

CONTROL ANALYSIS MODEL

The System Analysis Model was specially adapted to accommodate systemwide solution of the full St. Venant equations to enable detailed calculation of pipe filling, backwater, and pipe dewatering for evaluation of inline storage alternatives. In addition, control gates, wiers, orifices, and pump stations were coded which allowed for variation in operation based upon flows or water levels sensed either near or remote from the control structure. The model uses the regression flow model to calculate separate and combined sewer flows, then routes the flows through the complex sewer network and controls indicated in Figure 3. The model was used to calculate required control locations, gate and wier sizes and elevations, initial set points and system operation strategies.

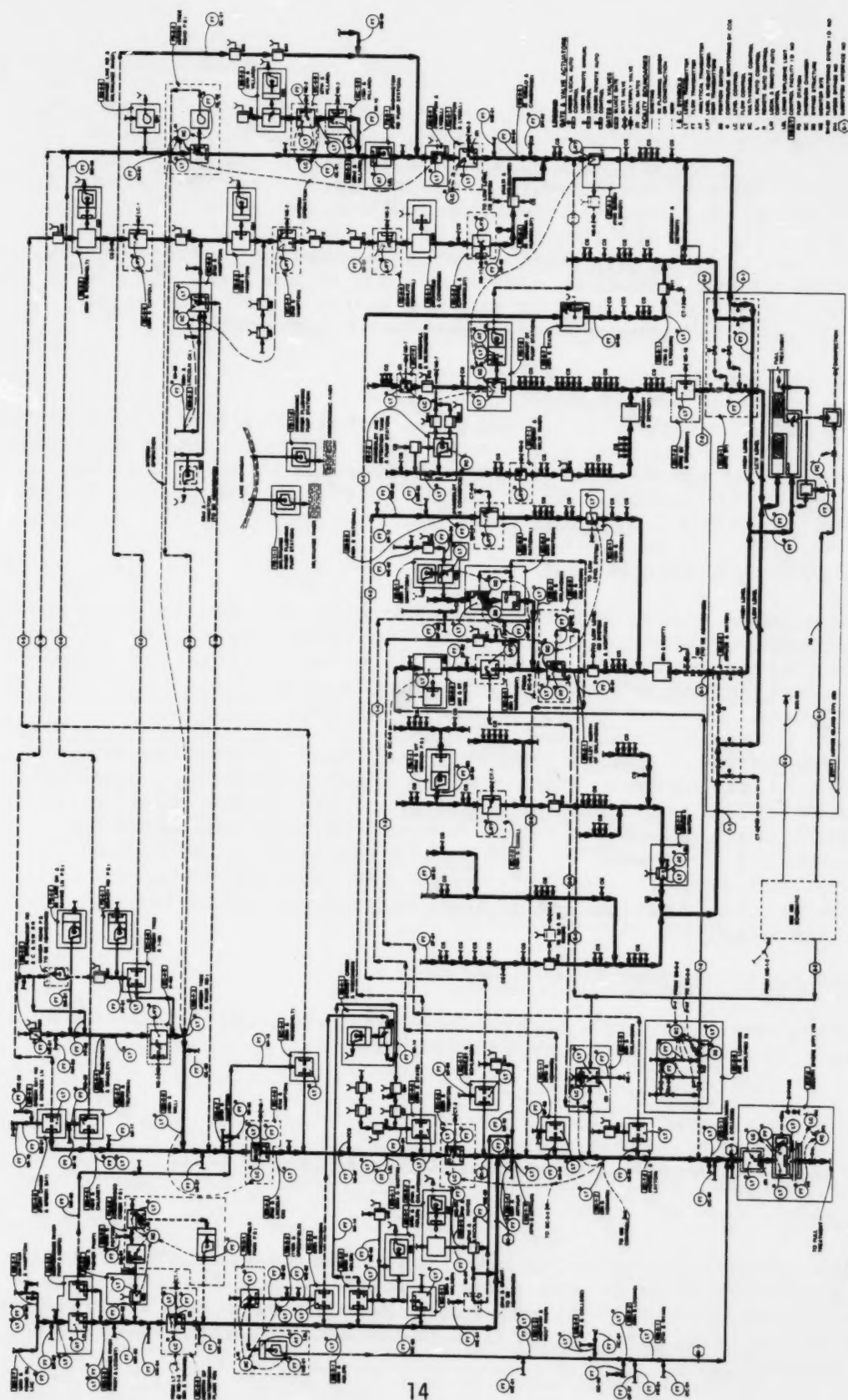
HYDRAULIC DESIGN

Transient hydraulic pressures anticipated in the tunnel system have been addressed through the use of:

- o CAM
- o SURGE
- o Transient model
- o Physical models of dropshafts

CAM

The previously used CAM model was used to evaluate filling characteristics of the tunnel system -- how fast it fills, which reaches fill first, etc. Results of these analyses were used to calculate air relief needs and evaluate transition hydraulics.



SURGE

SURGE is a water hammer program which was used to calculate pressures generated by sudden changes in flow rate in the surcharged portions of the inline tunnel conveyance/storage system. SURGE was also used for water hammer analyses in pressure lines elsewhere in the Milwaukee Metropolitan Sewer System.

TRANSIENT MODEL

A transient flow model developed at the University of Minnesota St. Anthony Falls hydraulic laboratory was utilized to evaluate the magnitude of transient pressures which might be expected during the transition from open channel to closed conduit flow in the conveyance/storage system.

PHYSICAL MODELS

Physical models of proposed vortex dropshafts were developed and evaluated at the University of Iowa hydraulic laboratory. These were used to evaluate alternative dropshafts configurations, establish scale factors, and estimate dropshaft design dimensions and pressures.

FUTURE MODEL USES

As indicated in the previous section, numerous water resources models have been required, developed, and used in the planning and design of the Milwaukee Water Pollution Abatement Program. The MMSD believes that it will have two types of uses for the computer models in the future. The first will be to use three of the models to assist in the operation and control of the District's proposed inline storage system. The second will be to use the Systems Analysis Model (SAM) for a variety of applications. These will include applications of the SAM model for analyses similar to those previously described and for data management. It has not been definitely determined that all the computer models will actually be used in the manner described. However, it is the intent of this paper to identify the various ways that the models could be used in the future. As the Milwaukee Water Pollution Abatement Program continues to develop, the extent of the continued use of the models will be determined.

OPERATION AND CONTROL

Three of the computer models have application to the control and operation of the inline storage system by using the models in a real time operation mode. These three models are the prediction model, CAM, and TUNNEL.

The prediction model can be used to aid in the prediction of separate sewer events that will require the use of storage. As discussed in the operation control analysis, the prediction model will use historic and predicted meteorologic data to determine the volume to be reserved for separate sewage.

Once the storage system is receiving flows from either the separate or combined sewer areas, the CAM model becomes useful for evaluating the filling of the system and responding to any changing conditions. For example, as the inline storage system is filling the tunnel volume occupied by wastewater and the influent flow rates at various points would be known. Given this information as inputs, the CAM model can be used to predict the effects on various parts of the tunnel system when the gates which allow combined or separated sewage into the system are modulated.

The third model, TUNNEL, becomes useful as the system is approaching full. Because gates cannot be closed instantaneously and because of the need to avoid surcharging of the tunnel system to avoid the possibility of exfiltration and resulting groundwater contamination, the system must be controlled such that it approaches its full capacity asymptotically. The TUNNEL model uses the volume of wastewater in the tunnel and the rate of flow approaching the gates as inputs to calculate which of the gates should begin closing first and how fast the gates should be closed to avoid surcharging of the tunnel system. TUNNEL algorithms will also assist in closing combined sewer inflow gates to reserve space for separate sewage.

SYSTEM ANALYSIS MODEL USES

As noted previously, the other potential future uses of the computer models developed for the Milwaukee Water Pollution Abatement Program center around the use of the system analysis model (SAM). The multiple uses of SAM stem from its ability to route a flow hydrograph through a sewer system to determine its effects on the downstream system. When some physical change is made to the system itself or to its service area, SAM provides a valuable tool for evaluation of the effects so that any resultant problems can be addressed and solved and not merely moved to a different location in the system. The future uses of the SAM model include:

- o continuing SSES evaluations
- o evaluation of future system expansion
- o analysis of combined sewer rehabilitation
- o data management

The MMSD has conducted an extensive sewer system evaluation survey on its tributary sewer systems which resulted in

various recommendations for sewer rehabilitation and relief which are now being implemented. However, it is recognized that such efforts will undoubtedly be needed on a continuing basis, though at a smaller scale. The SAM model could be used for continuing evaluation.

Additionally, land use and population changes will continue to occur in the service area. Even though the current program was planned to meet the needs of the planning area to the year 2005, there obviously are no guarantees that growth will occur at the same time or in the same patterns as predicted. Accordingly, new or different sized interceptor sewers may be required even before the year 2005 and almost certainly will be needed after that year. The SAM model will be used to evaluate the effects of such changes on the downstream sewer system. Such changes could potentially affect the diversion structures and operational control strategies for the inline storage system.

In addition to the uses by the Milwaukee Metropolitan Sewerage District, the model has application to the continuing combined sewer rehabilitation effort which has been ongoing within the City of Milwaukee for many years. Many parts of the combined sewer system approach and even exceed 100 years in age and have been sized using criteria far different from those used today. Because of this, the City of Milwaukee has had a long standing program of replacing portions of its combined sewer system each year and upgrading the sizes of combined sewers to meet the present day storm water conveyance criteria. Since the combined sewer area is approximately 27 square miles in size, there is potential in replacing selected sections of combined sewers to merely relocate a problem to a different area unless one has the ability to fully analyze the effects of such a change. Although such analyses can and have been done manually, the use of the model provides a faster and more definitive evaluation. This in turn permits a more optimally designed solution to any downstream problems created by the rehabilitation.

Finally, the model is useful for data management since it can serve as a library of information on the sewer system. Such data as diameter, slope, invert elevations, materials of construction, and general condition of a sewerline can be stored in the model's data base. Despite the fact that the District has a better than average set of system records, such data are more accessible and more easily retrievable if they are in computer files rather than only in plan files. Past experience has shown that retrieval of such information from a plan file can easily take 30 or more minutes of a technician or engineer's time while such data can be retrieved and displayed on a cathode ray tube (CRT) within a minute or two.

In summary, we believe that the District's Water Pollution Abatement Program has used state of the art storm water, water quality, and hydraulic computer modeling to a greater extent than probably any such municipal program in the country. Because much of the modeling effort conducted was specific to the Milwaukee Water Pollution Abatement Program, the models are expected to have continued uses to the District in the future and, with minor modifications and fine tuning may prove to be useful for as long as many of the physical facilities which will be constructed.

USING A TIMEX-SINCLAIR 1000 MICROCOMPUTER FOR REAL-TIME
CONTROL OF COMBINED SEWER OVERFLOWS

by

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ABSTRACT

Large quantities of combined sewage are diverted to Hamilton's receiving waters during storm events, significantly degrading the water quality. The pollution load can be lessened through more efficient flow control at the mechanized diversion structures in the city. The paper describes how a \$30 microcomputer may be used as a data logger and real-time controller. The computer communicates with on-site raingauges and flowgauges through a specially developed input/output interface. Software written in BASIC for the microcomputer processes incoming rainfall and predicts runoff at the diversion structure one step ahead in time using a simple discrete linear transfer function time-series model based on past rainfall and runoff. This rainfall-runoff model is developed from the output from a calibrated Stormwater Management Model, Version 3.2 (SWMM3), run in a continuous mode with a five-minute time-step. The microcomputer automatically controls the diversion structure so as to minimize the volume of combined sewer overflow, based on the forecast runoff. Flow forecast can be improved with the addition of an on-site flowgauge. Actual measurements can be used to correct any inaccurate forecasts, thereby ensuring that small errors do not accumulate. The microcomputer can handle hardware and software additions, increasing the level of control. The computer can be used to coordinate the operation of several diversion structures and gauges and/or nearby storage tanks simultaneously. Thus, city-wide control of diversion structures can be attained by implementing more complex rainfall-runoff time-series models using data from a network of several gauges to track a storm across the sewershed.

INTRODUCTION

Many major cities in North Eastern America are at least partly served by combined sewer networks which handle both storm and sanitary flows. During dry weather, all sanitary sewage travels to a sewage treatment plant (STP). During rainfall events, when stormwater runoff far exceeds the STP capacity, diversion structures in the combined sewer system divert excess flow, often directly to the receiving waters. The diverted flow includes sanitary wastewater. The paper discusses a methodology to reduce the impact of these combined sewer overflows (CSO). In particular, the feasibility of using a Timex-Sinclair 1000 microcomputer (TS1000) to control CSO diversion structures in real-time is investigated.



Figure 1.

Hamilton's sewer system presently contains mechanized diversion structures which might be controlled in real-time. We have chosen the combined sewer overflow structure located at Royal Avenue in West Hamilton (Figure 1). The contributing area measures approximately 20 sq. km. and is mainly comprised of single and multiple family dwellings, and open parkland.

Inside the regulator chamber at Royal Avenue, a motorized gate controls the entry of sewage into the mainline sanitary sewer which travels across the city. The gate is currently activated by a float recorder in the chamber but its operation is usually remotely overridden by staff at the Hamilton Wentworth Sewage Treatment Plant.

SCOPE OF THE STUDY

Real-time control (RTC) of diversion structures should be designed to:

- (a) make more efficient use of in-line storage,
- (b) prevent unnecessary or premature diversion of combined sewage,
- (c) divert unavoidable overflows to less sensitive receiving areas using a RTC system including several CSO structures,
- (d) control off-line storage facilities to store CSO until the sanitary interceptor can accommodate additional flow.

These strategies all allow more sewage to be treated at the STP.

Several North American cities have either implemented or recommended RTC systems. A computer augmented treatment and disposal (CATAD) system was proposed by Gibbs and Alexander (1969) for the Seattle Metropolitan area. Morrow and Labadie (1980) discussed an automated real-time control system for the city of San Francisco. McPherson (1980) investigated the "integral control of combined sewer regulators using weather radar", using Montreal as the study area. Henry and James (1981) proposed microcomputer control of combined sewer overflows in the Hamilton downtown area.

Until recently, RTC of hydrologic systems was effected by a large, expensive mainframe computer, usually centrally located with data telemetered from remote gauging stations. The stormwater models used were generally relatively complex. This scale of RTC system is still typical of river flood forecast warning systems which work on a one-hour (or larger) time-step. The time-step or lead time is necessarily much smaller for RTC of CSO, due to the rapid response time of urban catchments. A time-step of one to five minutes is more suitable.

SYSTEM HARDWARE

The system we have investigated includes the following components:

1. TS1000 microcomputer, manufactured in North America by Timex-Sinclair, and costing less than \$30 U.S.,
2. 16K RAMPACK for memory expansion,
3. Input/output interface for TS1000, produced in-house,
4. Drop counter precipitation sensor, produced in-house,
5. Pressure sensitive flow/depth gauge.

These components are all suitable for on-site installation at the diversion structure but in our study have so far only been tested in the laboratory. The TS1000 acts as the controller for the system, processing incoming observed rainfall and runoff data, and implementing the desired diversion strategy.

The TS1000 utilizes a Z80 microprocessor and is supplied with Sinclair BASIC in an 8K ROM. The basic TS1000 is supplied with only 2K RAM but is expandable to 64K. Sinclair produces a 16K expansion pack, but 32K and 64K extensions are available from many other hardware suppliers.

The small keyboard is covered by a plastic membrane. Rapid entry of coding on this keyboard is difficult but this problem is alleviated to some extent by Sinclair BASIC, whereby all BASIC keywords are entered by a single keystroke. The membrane keyboard is better protected from the damp environment of manholes than full-size standard, electro-mechanical keyboards.

Low-resolution graphics (44 x 64) is included. Graphics capabilities can be improved with hardware or software additions.

A cassette tape interface is included for easy loading and saving of programs but the transfer speed is relatively slow. Floppy disk interfaces are also available, and provide faster loads and saves.

The computer bus is accessible through the exposed edge connector at the back of the machine. This feature makes it easy to use the TS1000 as a data-logger and controller.

We have designed and built our own input/output interface which simply fits over the edge connector. The interface handles all incoming rainfall data and drives the relay for the motorized gate in the diversion structure. Three four-bit counters are cascaded for each gauge to log up to 4095 drops in each five-minute time-step. This translates to a maximum rainfall intensity of 6.00 in/hr. The counters are driven by the drop counter precipitation sensor (DCPS) developed at McMaster University (Haro,

Kitai and James, 1983).

Each set of counters is accompanied by a set of tristate buffers. These buffers control the input and output of information along the eight data lines. When enabled, a set of tristates allows the data on the corresponding counters to be retrieved by the TS1000. Sets of counters and buffers are enabled in sequence by the computer to collect the data from each DCPS.

The data lines are available for output when all the tristate buffers are disabled. The TS1000 uses the output portion of the interface to control a mechanized gate in the diversion structure.

A Timex 2040 thermal printer is used to plot hyetographs and tabulate the data for recorded storm events. The printer plugs onto the exposed edge connector in the same way as the I/O interface.

The 16K RAM pack, necessary to run our RTC system software, plugs onto a similar edge connector provided at the back of the printer or the I/O interface, depending on which is being used.

REAL-TIME CONTROL SOFTWARE

Programs may be written in Sinclair BASIC or Z80 machine code. Our software was written in Sinclair BASIC for simplicity. Machine code routines may be preferable in cases where computational speed is of great importance.

Our software has been titled "RTCONTROL". It was designed to control the automatic gate in the diversion structure at Royal Avenue in real-time. In order to effect this control, RTCONTROL must perform several tasks at once.

RTCONTROL handles all hydrological data input to the TS1000 through the specially designed I/O interface. Drop counts for each time-step are brought to the computer by polling the data lines every five minutes. The BASIC command to input the data from the interface is PEEK. A POKE outputs information to the interface. PEEK and POKE refer to the address in memory with which the interface communicates. In our case, this address is 8192. As an example, in order to retrieve the data, the following BASIC instructions are executed:

```
100 POKE 8192,1
110 LET N1=PEEK 8192
120 POKE 8192,2
130 LET N2=PEEK 8192
140 LET N=N1+256*N2
```

Because the interface uses a twelve-bit counter, and the TS1000 is only an eight-bit machine, data must be input in two steps. The first POKE enables the tristate buffers for the eight least significant bits of the drop count. These eight bits are then PEEKed into the TS1000. This procedure is then repeated for the four most significant bits of the drop count. The total number of drops is then calculated and converted to rain intensities (in/h.) for each five minute period. The final POKE clears all the counters to begin the next time step.

The TS1000 does not have a real-time clock built in but there is a facility for keeping track of elapsed time. The bytes in memory from 16384 to 16508 are set aside for specific uses by the system. They can be PEEKed to query the status of the system, and some can be POKEd to control the system. These system variables have names, but are referred to by their address only. FRAMES is a system variable which counts the number of frames displayed on the television monitor. The TS1000 works at a frequency of 60 Hz, thus displaying 60 frames per second. Bit 15 of FRAMES is 1. Bits 0 to 14 are decremented for each frame displayed. If 65535 is POKEd into FRAMES at the start of the simulation, it can then be PEEKed frequently to check elapsed time. FRAMES will return to 65535 approximately every 546 seconds, or 9 minutes. The timing routine in RTCONTROL recycles FRAMES and runs continuously until the program is terminated. The time-step used by RTCONTROL can be varied by the user.

Drop counts are POKEd into memory locations which are represented by a REM statement at the beginning of the program. Each drop count uses two bytes. The REM statement begins at location 16509, just above the system variables. Collected data thus becomes part of RTCONTROL and can be saved with the program. In order to save memory only non-zero drop counts and the number of dry time steps between these are stored.

After POKEing some data into the REM statement

```
5 REM XXXXXXXXXXXXXXXXXXXXXXXX
```

it might appear as:

```
5 REM $ = M +: / / 1+ 2 XX =
```

The drop counts which are POKEd into memory are represented by the printed character which has the character code equal to the drop count. Codes from 0 to 64 are used to store the rainfall data. Codes 156 to 186 are used to store the number of timesteps between non-zero drop counts. These appear as inverse characters in the REM statement. The last two characters in the REM statement identify the location in memory where the final measured drop count is stored.

The runoff hydrograph at the diversion structure, measured and/or forecasted, can be stored in memory in a similar manner.

TRANSFER FUNCTION MODEL

The computed rainfall intensities form one input time-series to the rainfall-runoff forecast model. As described later, an observed flow time-series may also be used as input to the model.

The rainfall-runoff process is represented by a discrete linear transfer function (TF) model of the following form:

$$Y(t) = \frac{\omega(B)}{\delta(B)} * X(t-b)$$

- where $\omega(b)$ = $\omega\phi - \omega_1*B - \omega_2*B^2 - \dots - \omega_S*B^S$
 $\phi(B)$ = $1 - \phi_1*B - \phi_2*B^2 - \dots - \phi_r*B^r$
 B = difference operator
 b = lag time between rainfall input and runoff response
 $X(t-b)$ = rainfall $I(t-b)$
 $Y(t)$ = runoff $Q(t)$
 R, S = order of the ω, δ terms

Identification of the model follows the methods outlined in "Time Series Analysis: Forecasting and Control", by Box and Jenkins (1976). Continuous rainfall records at five minute intervals form the input to the model with corresponding runoff time-series comprising the output.

Data is analyzed using the auto- and cross-correlation functions between the input and output. The process is assumed to be stationary if the auto- and cross-correlation functions of the $(X(t), Y(t))$ series damp out quickly. The series may need to be differenced to induce stationarity.

$$y(t) = (1-B)^d Y(t), x(t) = (1-B)^d X(t)$$

The degree of differencing, d is usually 0, 1, or 2. The orders of the model (r, s, b) are identified from the impulse and step response functions for the process.

Model residuals are calculated by substituting least squares estimates for the $w(B)$ and $d(B)$ parameters. If the auto- or cross-correlation functions of the residuals show marked correlation patterns the model is likely inadequate. These patterns may suggest the type of modification needed to fit an adequate TF model. An ARIMA type noise model may also be fitted to the residuals.

For identification purposes we chose to use as output the runoff time-series obtained from SWMM3 RUNOFF simulations using a five minute timestep.

Much work has been done on continuous runoff modelling in Hamilton (Robinson and James, 1981). Total pollutant loadings to Hamilton Harbour over the summer months from 1971 to 1978 were estimated by continuous SWMM2 and, more recently, SWMM3 using a one hour timestep. At present Robinson and James are investigating the introduction of variable time-step continuous modelling in SWMM3. Such a model would incorporate a five minute step during storm events and a one hour time-step during intervening dry periods.

Our current SWMM3 continuous model has been calibrated on a discrete event basis due to insufficient continuous quantity and quality records. However, calibration was completed using a five minute time step as required by RTCONTROL.

The model includes the entire west Hamilton area, draining to Cootes Paradise. Contained within this area is the Royal Avenue drainage basin.

The reasons for using computed rather than observed flows for the runoff time series are threefold:

1. Long-term continuous observed flow records at five minute intervals were not available due to occasional equipment malfunctions and breakdowns.
2. Using computed results reduces the need for field equipment; expensive gauging equipment is needed only for shorter calibration and validation data sets.
3. The operation of the RTC system may be checked by simulating the TF model and various diversion strategies using continuous SWMM3. In this way the performance of the RTC system can be evaluated from both a quantity and quality standpoint. Important quality parameters used include PO4, NIT, settleable solids, and BOD5.

Initial identification of a rainfall-runoff TF model for the larger basin proved quite successful. This identification used actual flows and was done on several of the more significant discrete events:

$$Y(t) = \frac{9.55 + 13.43*B}{1 - 0.93*B} * X(t-4)$$

or:

$$Y(t) = 0.93 * Y(t-1) + 9.55 * X(t-4) + 13.43 * X(t-5)$$

The TF model was validated for the storm of July 22, 1982 using the rainfall record from the gauge at McMaster University and the hydrograph measured at the

Hydro Substation site. Results are given in figure 2.

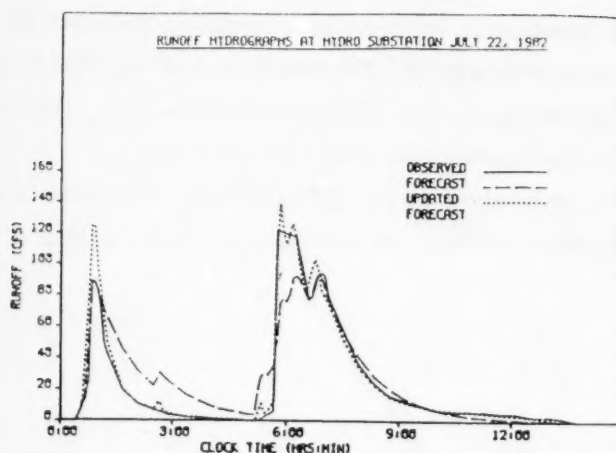


Figure 2.

The TF forecast model performs fairly well when compared to recorded flows. The timing and magnitude of the peaks are quite accurate. However the receding limbs of the hydrograph are not adequately represented. This presents a problem in determining the proper time to reopen the gate. If the flows on the falling limb of the inflow hydrograph are overestimated overflow to the receiving waters will likely be above desirable levels.

Additional parameters were included in the TF model but proved to be insignificant. The addition of real-time flow readings to the RTC system can vastly improve the TF model's forecasting capabilities. Real-time flow measurement can be substituted for $Y(t-1)$ in the TF model, correcting the previous forecast. Figure 2 shows a marked improvement in the receding portion of the hydrograph. Predicted runoff volumes are much more accurate.

DIVERSION STRATEGY

Based upon the forecast of runoff at the diversion structure RTCONTROL effects operation of the automatic gate controlling the flow of combined sewage to the local treatment plant and receiving waters. The exact level at which the gate is to be opened and closed is decided using SWMM3 continuous models which incorporate the rainfall-runoff TF model and diversion strategies. Various diversion schemes can be evaluated in terms of receiving water quality.

The chosen diversion strategy is programmed into the TS1000 which controls the

gate through the I/O interface.

DATA PROCESSING

RTCONTROL is completely interactive, prompting the user for input regarding the TF model and diversion strategy and displaying all hydrological information in real-time. Used in the control mode the program collects data, stores it in the REM statement, and operates the diversion structure in real-time.

Used in the data processing mode, RTCONTROL retrieves the data from the REM statement and produces listings and hyetographs of the events on the Timex 2040 printer (figure 3).

GAUGE LOCATION = BOND AVENUE
START TIME = 0000
PEAK INTENSITY = 2.11 IN HR
STORM 21, AUGUST 2, 1982

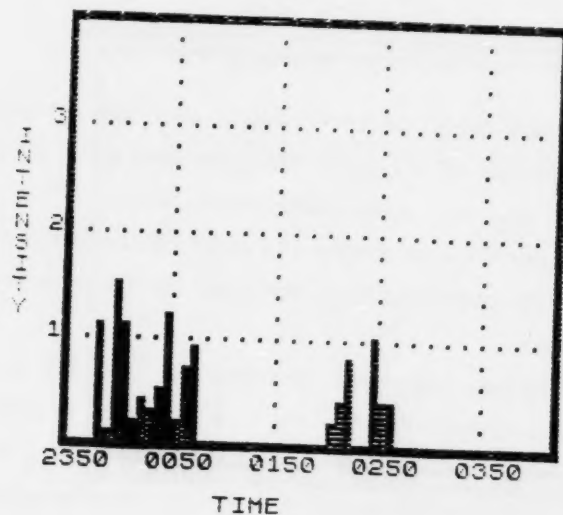


Figure 3.

The full version of RTCONTROL currently logs approximately three days of continual rainfall at five minute intervals using a 16 K RAM pack. The field version of RTCONTROL requires much less memory than the full version because it is not interactive nor does it print to the monitor. This allows at least one month of continual rain to be logged for one gauge.

CONCLUSIONS

A system for controlling combined sewer overflows in real-time has been

designed and tested using a Timex/Sinclair 1000 microcomputer. Rain gauges collect data in real-time and communicate with the TS1000 through a specially developed input/output interface, all installed on-site at the diversion structure.

The software which runs the RTC system is written in BASIC. The program is named RTCONTROL and accomplishes the following tasks:

- 1) counts drops in programmable time intervals using a special sensor,
- 2) computes rain intensity from drops counted,
- 3) logs rain data,
- 4) forecasts flows using a transfer function model to represent the rainfall/runoff process,
- 5) outputs a signal to drive the relay for a mechanized gate,
- 6) plots hyetographs on TS2040 printer and lists data.

The full, interactive, version of the program currently logs about three to four days of continual data at five minute time steps. The stripped down field version of RTCONTROL allows at least one month of continual rain to be logged for one gauge.

The system performs well in the lab and we foresee no problems with its field installation in the near future. The rainfall-runoff transfer function forecast model provides fairly accurate predictions of runoff one step-ahead in time. Its performance is further improved with the addition of real-time flow measurements to the basic system configuration. Recorded flows can be used to correct for inaccurate TF model forecasts.

Ongoing work includes:

- 1) modification of input/output interface to include an analogue to digital converter to log streamflow data,
- 2) use of more sophisticated time-series models to represent the rainfall-runoff process; including multivariate models and/or kinematic storm modelling,
- 3) development of a variable time-step option for SWMM3 continuous model,
- 4) definition of measures of effectiveness for the system using SWMM3 continuous models incorporating the rainfall-runoff transfer function model and diversion strategies.

The TS1000 microcomputer provides an efficient means of collecting data and controlling combined sewer overflow structures in real-time. This system is easily implemented and very inexpensive.

REFERENCES

- Box, G.E.P. and Jenkins, G.M. (1976). Time Series Analysis: Forecasting and Control. 575 pp.
- Gibbs, C.V. and Alexander, S.M. (1969). CATAD System Controls for Regulation of Combined Sewage Flow. Water and Wastes Engineering, Vol. 6, No. 8, pp. 46-49.
- Haro, H., Kitai, R. and James, W. (1982). Precipitation Instrumentation Package for Sampling of Rainfall. Institute of Electrical and Electronics Engineers (Transactions on Instrumentation and Measurement). Vol. IM32, No. 3, pp. 423-429.
- Henry, D. and James, W. (1981). Investigation of Management Alternatives for Combined Sewer Overflows for the City of Hamilton. Proceedings of the Conference on Water Quality and Stormwater Management Modelling. Niagara Falls, Ontario. U.S. EPA, p. 493-512.
- Robinson, M.A. and James, W. (1981). Continuous SWMM Quality Modelling for the City of Hamilton using Atmospheric Environment Service Data. Proceedings of the Conference on Water Quality and Stormwater Management Modelling, Niagara Falls, Ontario, U.S. EPA, pp. 469-492.
- McPherson, M.B. and Ammon, D.C. (1980). Integrated Control of Combined Sewer Regulators using Weather Radar. Municipal Environmental Research Laboratory. Office of Research and Development, U.S. EPA, 87 pp.
- Morrow, D.M. and Labadie, J.W. (1980). Urban Stormwater Control Package for Automated Real-Time Systems. Proceedings of the Canadian Hydrology Symposium: 80 - Hydrology of Developed Areas. NRC of Canada, pp. 28-39.
- Vickers, Steven (1982). Timex User Manual - Timex Sinclair 1000. Timex Corporation and Sinclair Research Limited, 154 pp.

HYDRAULIC MODELING WITH SWMM IN AN
UNSTEADY PRESSURE FLOW REGIME

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APRIL, 1984

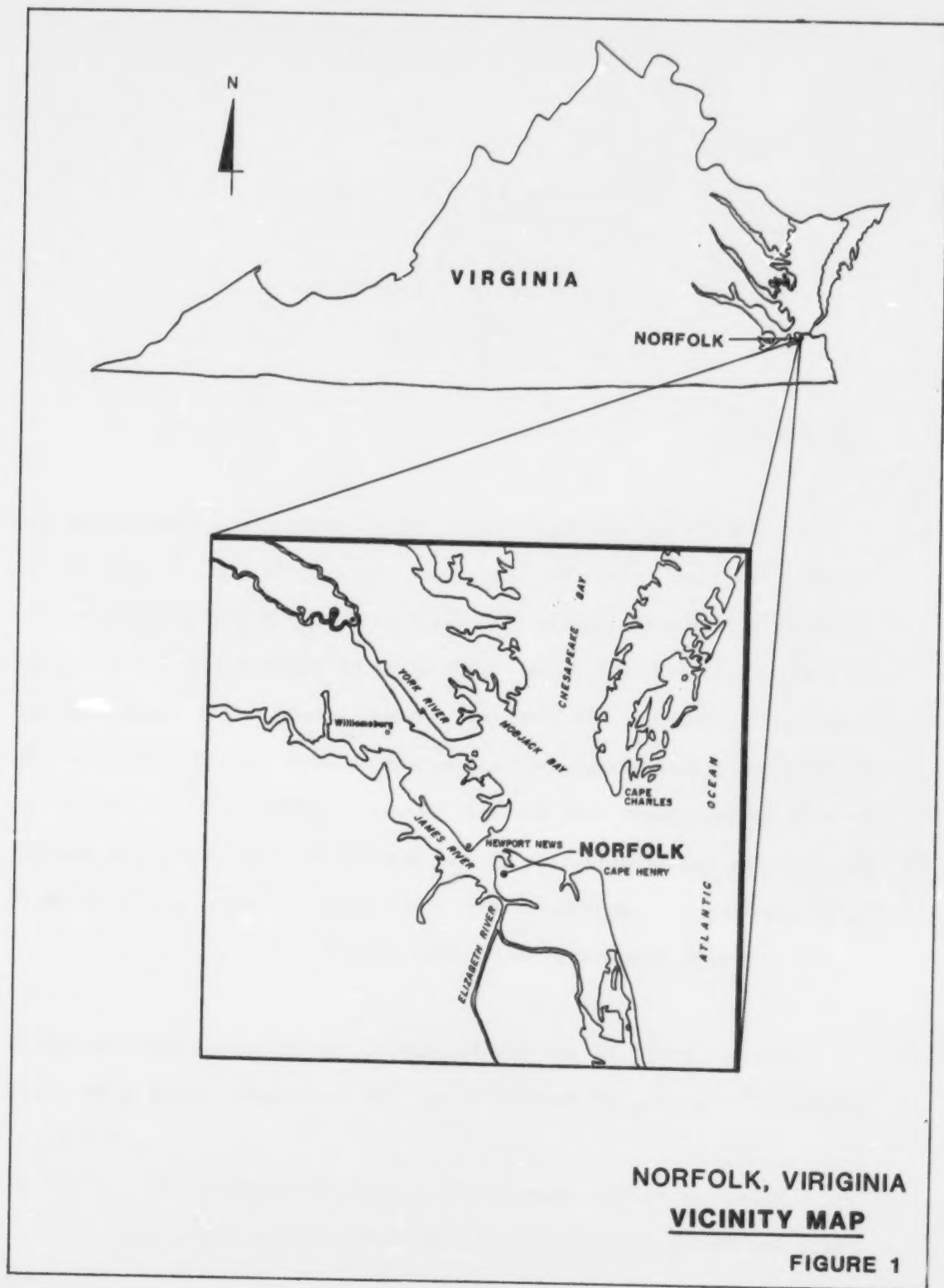
INTRODUCTION:

The Monticello Drainage Basin (MDB) comprises approximately 480 acres of the City of Norfolk, Virginia. Located on the Elizabeth River, a tributary of the Chesapeake Bay (Figure 1), this highly urbanized basin is subject to tidal tailwater influences in addition to limited topographical variations. In the last several years, major commercial and residential redevelopment has taken place under the direction of the Norfolk Redevelopment and Housing Authority (NRHA). As is typical of many cities, the urbanization of the Monticello area of Norfolk has led to a high degree of impervious land cover which, in turn, has resulted in a much increased quantity of stormwater runoff.

Installed prior to the redevelopment, the MDB storm sewer system is presently incapable of accommodating the increased runoff from storm

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events which approximate a one-year frequency. Many parts of the Basin are subject to flooding several times per year, due to the inadequacies of the drainage network. Hence, the Monticello Drainage Study was conceived to develop phased stormwater system improvements such as would handle a more acceptable storm event.

In the summer of 1983, the Department of Public Works of the City of Norfolk authorized CE Maguire, Inc. to study the MDB and recommend improvements to the existing drainage system which would alleviate street-flooding during the coincidence of a two-year storm event and a mean high tide (elevation 102.3 Norfolk City Datum) as the tailwater at the outfalls. The MDB was addressed in this study as a high priority region, requiring major improvements to reduce or eliminate flooding, which has been the cause of private property damage and severe traffic congestion.

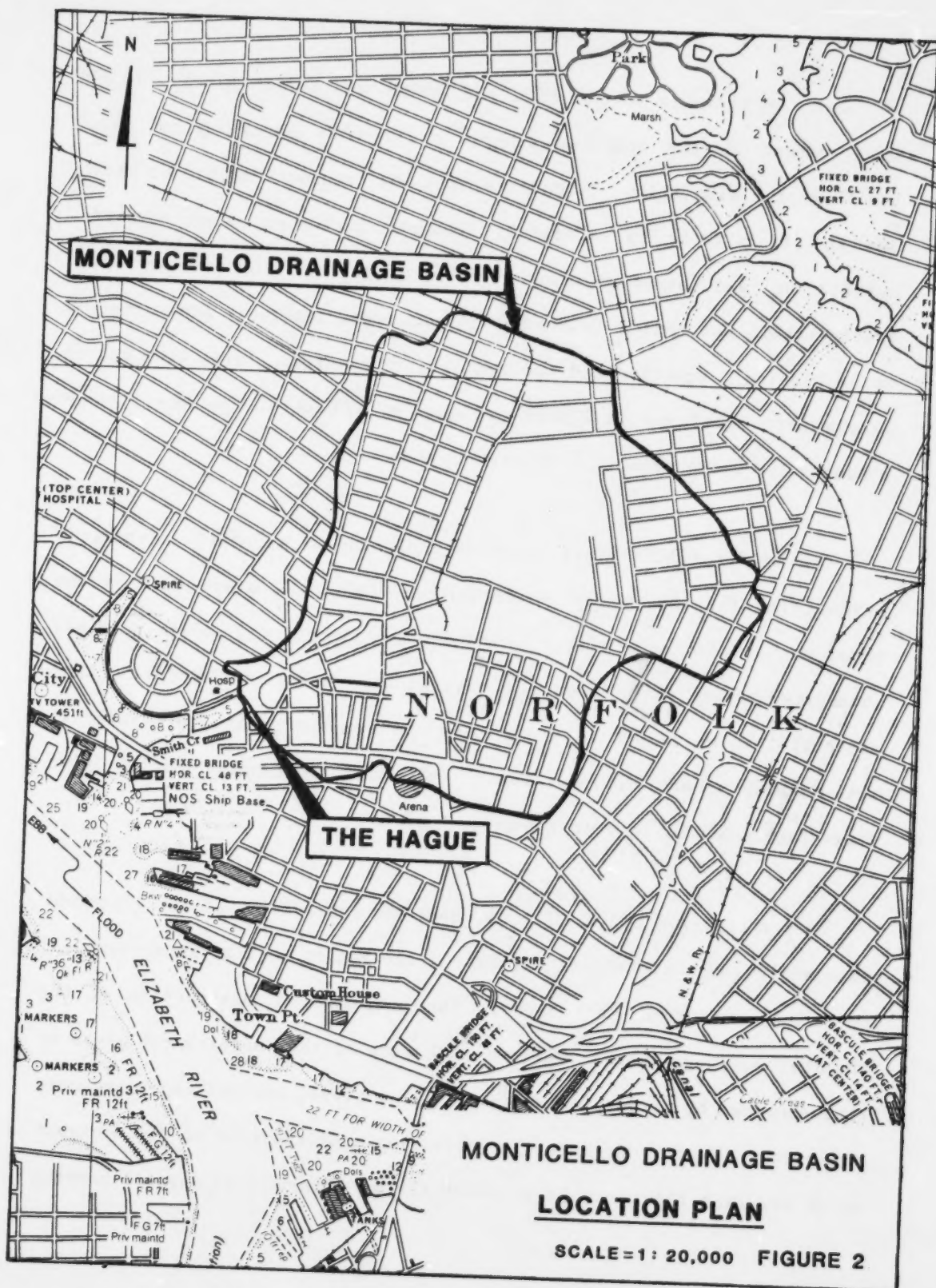
In order to evaluate the total urban rainfall-runoff process for an area as large as the MDB, it was recognized that a comprehensive mathematical computer simulation model would be required. The model must have been able to assimilate an accurate depiction of the typical system and also provide an opportunity to evaluate alternate flood abatement procedures. Due to the nature of the existing conduit network and the constant tailwater influence, it was imperative that the model of choice be capable of simulating parallel pipes, looped systems and, perhaps most importantly, surcharge within the system. The Stormwater Management Model (SWMM) Version III, August, 1983 release was selected. The

Extended Transport Module (EXTRAN), considered to be one of the most sophisticated computer programs available in the public domain for detailed hydraulic analysis of storm water systems, permitted the flexibility required for this analysis.

THE MONTICELLO DRAINAGE BASIN (MDB):

The 480 acre MDB (Figure 2) drains generally to the south with the basin outfall discharging into The Hague on the eastern branch of the Elizabeth River. During periods of high tide, the outfall conduits are completely submerged and a large percentage of the available pipe storage is occupied by tidal waters. The existing stormwater system in the MDB consists of a series of circular conduits ranging in size from 6" to 66" in diameter and box culverts as large as 48" x 72". This network is comprised of three major trunk lines identified as the Northern, Central and Southern Trunks, all three of which are confluent at the outfall system.

As was stated earlier, much of the basin's land cover is impervious. Major commercial and residential redevelopment has increased the degree of imperviousness to approximately 40 percent of the entire MDB land area. The resulting increased quantity of storm runoff, coupled with the generally low-lying nature of this coastal drainage basin, has made the antiquated stormwater network obsolete.



For the analysis of the MDB, the RUNOFF and EXTRAN Blocks of SWMM were adopted as the computer program arrangement necessary for total runoff system simulation. The latter was the transport method of choice due to its ability to accurately simulate backwater, flow reversal, surcharging and pressure flow. Thus, overland flow derived from precipitation was routed in RUNOFF by non-linear reservoir approximation using Manning's equation and the continuity equation. Inlet flow hydrographs were developed in RUNOFF and used by EXTRAN to simulate closed-conduit flow via a dynamic flow routing model.

Use of the RUNOFF block required the input of rainfall intensities versus time for the period of simulation. The hyetograph used in the model was a two-year six-hour duration storm event. Initially, rainfall intensity values were input for five-minute intervals (THISTO). Subsequently, in an effort to "smooth out" hydrograph peaks, this interval was reduced to one minute. These revisions effectively reduced inlet hydrograph peaks by five to twenty percent.

Discretization of the MDB included the subdivision of the catchment into sixty (60) subcatchment areas. These ranged in size from three (3) to nearly seventeen (17) acres. The mathematical abstraction of the physical drainage system required that each of the subcatchment areas be characterized by various parameters such as slope, percent imperviousness and width. "Width" is a fairly subjective approximation of the distance which overland flow travels to reach the main conduit or channel passing

through a subcatchment. Table 1 enumerates the range of values assigned to the above-mentioned parameters.

TABLE 1
Variable Subcatchment Parameters

<u>Parameter</u>	<u>Low</u>	<u>High</u>
Slope (Ft/Ft)	0.003	0.02
Imperviousness (%)	5	80
Width (Ft)	100	1,600

In addition to the above, several parameters were assumed constant over the entire sixty-catchment area. These constants are presented in Table 2.

TABLE 2
Constant Subcatchment Parameters

<u>Parameter</u>	<u>Value</u>
Manning's "N":	
Impervious Area	0.013
Pervious Area	0.27
Depression Storage (in):	
Impervious Area	0.30
Pervious Area	0.35
Green-Ampt Infiltration Parameters:	
Capillary Suction (in)	10.00
Hydraulic Conductivity of Soil (In/Hr)	0.1
Initial Moisture Deficit (Vol. Air/Vol. Voids)	0.32

The conduit network was defined in EXTRAN as a series of links and nodes. Inlet structures were identified as nodes to accommodate inflow from RUNOFF. In addition, junction and outfall structures were utilized to define conduit termini. For each structure, rim and invert elevations were established. The rim (ground) elevation at each structure served as an upper boundary to the hydraulic gradient, exceeding this limit resulted in "street flooding". The EXTRAN model is very sensitive to the time-step integration period (DELT). If this increment is too large, flows in the conduit system cannot balance and an oscillating network results. To avoid this situation, the time-step must approximate the wave celerity in the system, and is estimated by the following equation:

$$\Delta t_c = \frac{L}{\sqrt{gD}} \quad \text{where } \Delta t_c = \begin{array}{l} \text{time for a surface wave to travel} \\ \text{from one end of a conduit to the} \\ \text{other (seconds)} \end{array}$$

L = Conduit Length (feet)

g = 32.2 (ft./sec²)

D = Channel depth or pipe diameter (feet)

For simulation of the MDB, discretization resulted in the lengths of modeled conduit ranging between one hundred (100) and five hundred (500) feet. Early attempts to utilize a 15 second time-step (Δt_c) produced erroneous results. It was necessary to set the time-step at 10 seconds in order to accommodate the shortest conduits in the system.

The terminus of the schematized existing system consisted of four (4) free outfalls at The Hague. Tidal backwater conditions were simulated assuming a constant stillwater level (SWL) of 102.3 feet Norfolk City Datum (NCD), which is approximately mean high water (MHW). This tidal elevation was projected into the conduit system by initializing water depths at junctions which experienced tidal inflow.

APPLICATION OF THE MODEL

As stated previously, the two SWMM blocks utilized in this analysis were RUNOFF and EXTRAN. The TRANSPORT block could not be effectively used due to the significant degree of surcharging in the existing system. The overall objectives of this modeling process were as follow:

1. To model the existing hydrologic and hydraulic conditions in the basin and to calibrate the model with historic flooding information.
2. To evaluate alternate flood abatement measures.

Due to the many conduits and branches in the system, the initial development of the link and node pattern was a critical aspect of the overall process. Since the EXTRAN improvement runs would consider looping and cross-connecting the northern, central and southern trunks, the entire basin had to be modeled simultaneously rather than modularly. The total number of links and nodes in the system became a very serious

concern, since EXTRAN limits the user to a maximum of 187 conduits and 187 junctions.

For the existing conditions, the storm sewer network was described with 140 links and approximately 110 nodes. This network went through a very rigorous process of verification and calibration. As is typical of computer simulation, the initial runs displayed a high degree of balancing instability and oscillating flow rates. Typically, these problems were solved by reducing the computational time-step (DELTA) to 10 seconds and adjusting the lengths of the shorter links in the system with the equivalent pipe method. Both of the methods are discussed in the User's Manual for EXTRAN (Roesner et al, 1982) and are clearly presented and well documented.

Through further review of the output, a dramatic hydraulic continuity imbalance was discovered in each of the conduits classified as a rectangular section. Due to the large quantity of reinforced concrete box sections in the model, this problem had very serious ramifications. The continuity imbalance was detected in the output table described as "Summary Statistics for Conduits". Typical information and data shown on this table include the conduit numbers and vertical depth, the maximum computed flow and velocity, and the maximum water depths above the inverts. During a period of time when a conduit is surcharged, a very simple continuity check (i.e., $Q = V \times A$) can be performed with the information on the summary table and with the known cross-sectional area of the full-flowing conduit. The maximum computed flow was divided by

the cross-sectional area in order to establish an estimate of the maximum computed velocity. For circular and arch pipes the estimates were consistently within 5 percent of the expected values and were generally within 1 percent. On the other hand, the velocity estimates for the rectangular sections were between 10 and 255 percent greater than the maximum computed velocities from the program. Table 3 presents a sample of the continuity imbalances encountered. All of the conduits listed on the table are in a surcharged condition. Conduit type number 2 refers to a rectangular section and type 5 refers to a basket-handle cross-section.

TABLE 3

Rectangular Section Continuity Imbalance

(1) CONDUIT NUMBER	(2) CONDUIT TYPE	(3) SIZE H X W (FT)	(4) AREA (FT ²)	(5) MAXIMUM COMPUTED FLOW (CFS)	(6) MAXIMUM COMPUTED VELOCITY (FPS)	(7) VELOCITY 5/4 (FPS)
13821	2	3.5 x 5.0	17.5	156.9	5.1	9.0
	5	3.5 x 5.0	17.5	68.9	3.9	3.9
13398	2	3.5 x 5.0	17.5	89.3	4.2	5.1
	5	3.5 x 5.0	17.5	73.8	4.2	4.2
13665	2	3.0 x 4.0	12.0	36.9	2.8	3.1
	5	3.0 x 4.0	12.0	29.9	2.5	2.5
13366	2	1.5 x 5.0	7.5	125.1	4.7	16.7
	5	1.5 x 5.0	7.5	69.3	9.7	9.2
13369	2	3.5 x 5.0	17.5	128.3	4.9	7.3
	5	3.5 x 5.0	17.5	80.7	4.6	4.6

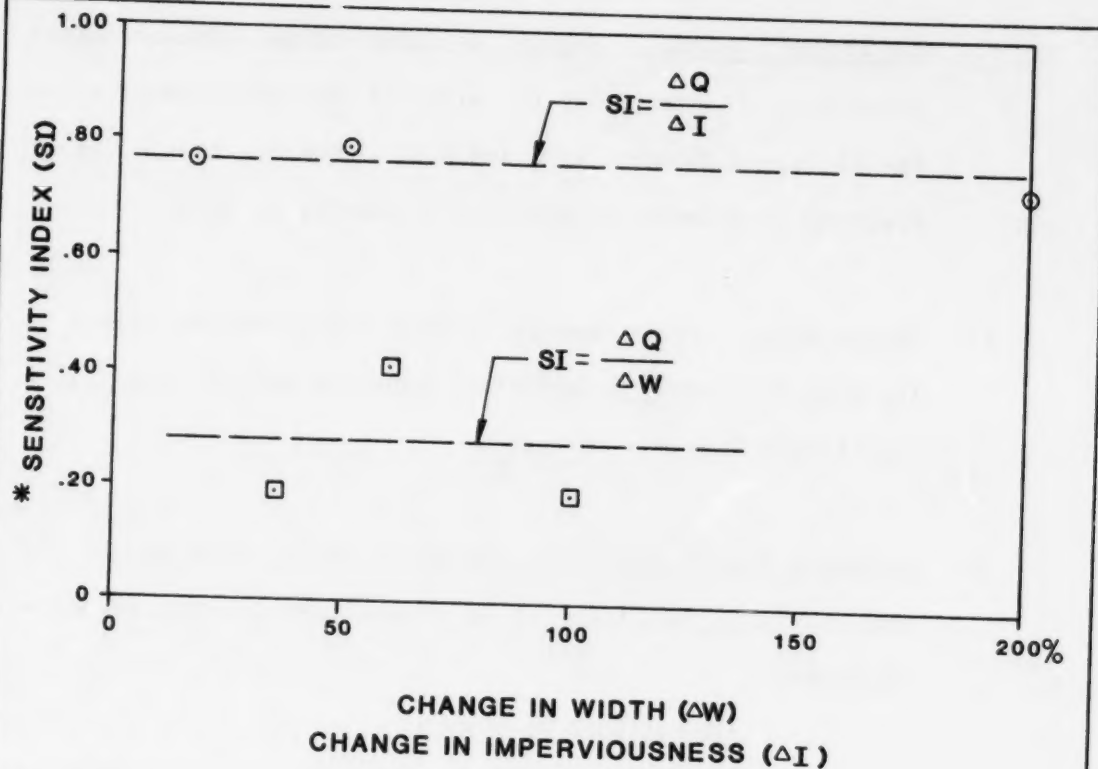
In an attempt to identify the cause of the problem the input data was scrutinized but no errors were found. Following the input review,

all of the rectangular sections were replaced by basket-handle sections in an effort to rectify the imbalance problem. Results from the subsequent EXTRAN run show that the continuity imbalance was rectified. Table 3 also shows the comparative results for the rectangular versus basket-handle sections for selected conduits. At this point in time (April, 1984), it is not known if the problem with rectangular conduits lies with the compiled version of SWMM used by the authors, or if it is universal to the source tape.

With the existing condition model being debugged and functioning properly, the process of calibration could then begin. Calibration involved both the RUNOFF and EXTRAN blocks simultaneously. In this way, the effects of the runoff calibration parameters were observed right through the storm drainage network. This was advantageous since the calibration was being geared towards matching areas which are known to be historically prone to flooding.

Parameters used to calibrate the model are listed below in descending order of sensitivity:

- a) Percent Impervious: As shown on Figure 3, the sensitivity index (SI) for a change in percent imperviousness is about 0.8 over the range of values tested. Peak discharge in RUNOFF was, therefore, found to be very sensitive to revisions in the percent of impervious area.



* "SENSITIVITY INDEX" (SI) IS A MEASUREMENT OF THE RELATIVE CHANGE IN PEAK FLOW (ΔQ) WITH RESPECT TO EITHER THE CHANGE IN SUBCATCHMENT "WIDTH" (ΔW) OR THE CHANGE IN "IMPERVIOUSNESS" (ΔI): $SI = \frac{\Delta Q}{\Delta W}$ OR $SI = \frac{\Delta Q}{\Delta I}$

SENSITIVITY INDEX

FIGURE 3

- b) Subcatchment Width: Figure 3 also shows the relative sensitivity of changes in the width of the subcatchment area. The SI ranged between 0.15 and 0.41 indicating that the peak discharge is moderately sensitive to changes in width.
- c) Ground Slope: Slope changes created only minor variations in the peak discharge. Even 10-fold increases did not result in a significant flow rate change.
- d) Roughness Factor, Depression Storage & Infiltration Rates: The peak discharge was found to be insensitive to each of these parameters.

After the model was calibrated to assimilate the physical realities of the basin, alternate improvements were investigated in order to determine the most cost-effective solution. Abatement methods included the following options:

- a) Tide gates at each outfall;
- b) Tide gates in conjunction with pumping;
- c) Expanding the hydraulic capacity of the system by adding new conduits.

The application of tide gates to the model, along with various assumed initial water elevations within the system was not as simple as indicated in the User's Manual (Roesner et al, 1982). For the basin

being modeled, many of the 150 junctions had invert below the tide level. Since at this point in time there is no way of simultaneously initializing the starting water surface elevation at all of the junctions, the procedure must involve a review of each junction elevation and adjustment of the initial depths.

When the existing system was expanded to include all of the modifications and improvements for the 2-year rainfall event, the limits of EXTRAN's capacity were nearly attained. The network included 182 links and 150 nodes with 187 being the maximum for each.

CONCLUSIONS AND RECOMMENDATIONS

The SWMM Version III model was successfully utilized to simulate a large, highly developed drainage system. The entire conduit system was modeled with EXTRAN due to the surcharging environment.

Several shortcomings in the EXTRAN block, which caused difficulty from the user's point of view, are shown below.

1. Conduits could not be classified as rectangular. Erroneous results occurred with rectangular sections.
2. Starting junction water depths could not be determined by the program even if the system were to start at a constant elevation.

Recommendations include a review of the SWMM Version III, August 1983 program to determine if a problem exists with rectangular conduit shapes in EXTRAN. As the SWMM program is updated, it would be desirable to include the capability of determining starting depths at junctions when a constant elevation is desired. When a constant elevation is not appropriate, the program could then default to the present method or vice-versa.

LIST OF REFERENCES

- Aldrich, J.A., and Roesner, L.A. (1982), "An Improved Surge Computation in EXTRAN", Proceedings of Stormwater and Water Quality Management Modeling Users Group Meeting, Washington, D.C., March, 1983 (EPA-600/9-81-015, August, 1982).
- CE Maguire, Inc. (1984), Monticello Storm Drainage Improvements: Computer Analysis and Drainage Study (Preliminary Draft), Virginia Beach, Virginia.
- Huber, Wayne C. (1983), "The EPA Storm Water Management Model: Documentation and Maintenance", Proceedings of the Conference on Frontiers in Hydraulic Engineering, Cambridge, Massachusetts, August, 1983.
- Huber, Wayne C.; Heaney, James P.; Nix, Stephan J.; Dickinson, Robert E.; and Polmann, Donald J. Storm Water Management Model User's Manual-Version III, Gainesville, Florida, Fifth Printing: January, 1983.
- Parker, W.M., III (1980), "The Use of SWMM to Economically Model Surcharged Combined Sewer Systems", Proceedings Stormwater Management Model Users Group Meeting, Gainesville, Florida, January, 1980 (EPA 600/9-80-017, March, 1980).
- Roesner, Larry A., Shubinski, Robert P., and Aldrich, John A. Stormwater Management Model User's Manual Version III: Addendum I EXTRAN, Annandale, Virginia, Fourth Printing: October, 1982.

A Lake Acidification Model Using WASP

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Introduction

The purpose of this modeling study is to develop a lake acidification model which is capable of predicting pH and alkalinity levels in a lake on a seasonal basis. In general, two different modeling approaches can be found among existing models. Simplified, empirical methods or models have been developed using historical lake chemistry data from lakes receiving acid inputs. These simplified models are based on the equilibrium of chemical constituents in the lake water. They are often used to identify lake acidification and to estimate the degree of acidification. The models or methods fall into this category include the excess sulfate method (16), pre-acidification alkalinity method (17), alkalinity vs. calcium and magnesium plot (22), pH vs. calcium model (16), and calcium and magnesium vs. sulfate model (17). These simplified models or plots, while interesting in describing the general trends in lakes, are too crude to permit a quantitative analysis of the effects of acid input on lake water quality in terms of pH and alkalinity. As a result, their predictive capability is highly limited because inferring acidification or the reverse of acidification are not accounted for in the models.

Another approach utilizes a more complicated representation of the entire watershed system and lake basin than the simplified modeling approach. It incorporates interactions between the watershed and lake water. Such an approach requires a significant amount of data to characterize numerous watershed hydrology and neutralization factors as well as physical, chemical, and biological processes in the lake. This approach, with sufficient data, is designed to quantify a cause-and-effect relationship between the acid deposition rate and the lake's response in terms of explicit calculations of alkalinity, pH, and major cations and anions in the lake water. This is the approach adopted in this study to develop a lake acidification model.

Proposed Model and Major Mechanisms

While many integrated watershed-lake acidification models address the terrestrial and aquatic systems, the model developed focuses on the water column in a lake or impoundment. Such a simplification is not intended to discount the importance of the land system. Instead, input from the land or watershed is incorporated as input data for the lake acidification model.

Model Variables -- The primary acidification indicators addressed in this model are alkalinity and carbon dioxide acidity. Understanding the carbon dioxide-bicarbonate-carbonate equilibria in natural water systems (30) indicates that alkalinity, carbon dioxide acidity, and pH are the three most important water quality indicators in the lake acidification process. Usually, the pH of a natural water system can be determined once alkalinity and carbon dioxide acidity levels are known. Thus, alkalinity, CO_2 acidity and pH are considered the primary water quality variables in the model.

Lake Hydrology and Transport -- Lake conditions vary seasonally during an annual cycle. One of the important seasonal phenomena is temperature stratification. For example, during the summer months, limited exchange of mass and heat energy occurs in the vertical direction because of stratification. As a result, significant concentration gradients (alkalinity, for example) may result. The degree of vertical resolution needed to address the concentration gradients is primarily determined by the extent of data available to quantify the vertical exchange of mass. A two-layer resolution is adopted in the model to approximate the vertical concentration gradient in the water column. The surface layer is referred to as the epilimnion and the bottom layer as the hypolimnion of the lake. In addition to internal transport, interactions between the lake and its watershed are also included. For example, tributary flows to the lake and lake outflows via outlets are included. The general quantification of the mass flux associated with these flows is in the form of mass loading rate which equals the product of concentration and flow in the tributary. Ground water flow into the hypolimnion is also incorporated.

Air-Water Interface Exchange -- Carbon-dioxide can be transported across the air-water interface and therefore affects the pH levels in the water column. The direction of this transport depends upon whether the epilimnion of the lake is undersaturated or oversaturated with respect to CO_2 . Thus, a gain or loss of CO_2 due to this process is quantified as a gain or loss of CO_2 acidity in the epilimnion. A similar mechanism for alkalinity does not exist at the air-water interface. Thus, alkalinity, as defined in standard chemical terms, does not change in value with a gain or loss of dissolved CO_2 . In other words, CO_2 variations do not alter the charge balance of the system that is, in turn, reflected in the alkalinity value.

Biochemically Mediated Processes -- The carbonate system is not the only buffering system operative in lakes or impoundments against acidification. For example, photosynthesis can be looked upon as a net consumer of hydrogen ions. Hypolimnetic sulfate reduction during period of anoxia will increase the buffering capacity by decreasing the total charge on nonprotolytic anions (i.e. SO_4). On the other hand, the nitrification reaction, in which ammonia is oxidized to nitrate, decreases alkalinity (i.e. H^+ is produced). Weber and Stumm (31) summarized a number of biochemically mediated processes affecting the pH and alkalinity levels in natural waters (see Table 1). The net result of these processes may be significant in neutralizing acidic input to some lakes. Schindler (25) reported that in Lake 223 of Experimental Acidification in Canada, over 60% of the acid neutralization was accounted for by sulfate reduction and denitrification. Thus, it is important to incorporate the buffering mechanism offered by some of the biologically mediated processes into the modeling framework.

Table 1. Biologically Mediated Processes (affecting pH and Alkalinity) in Lakes

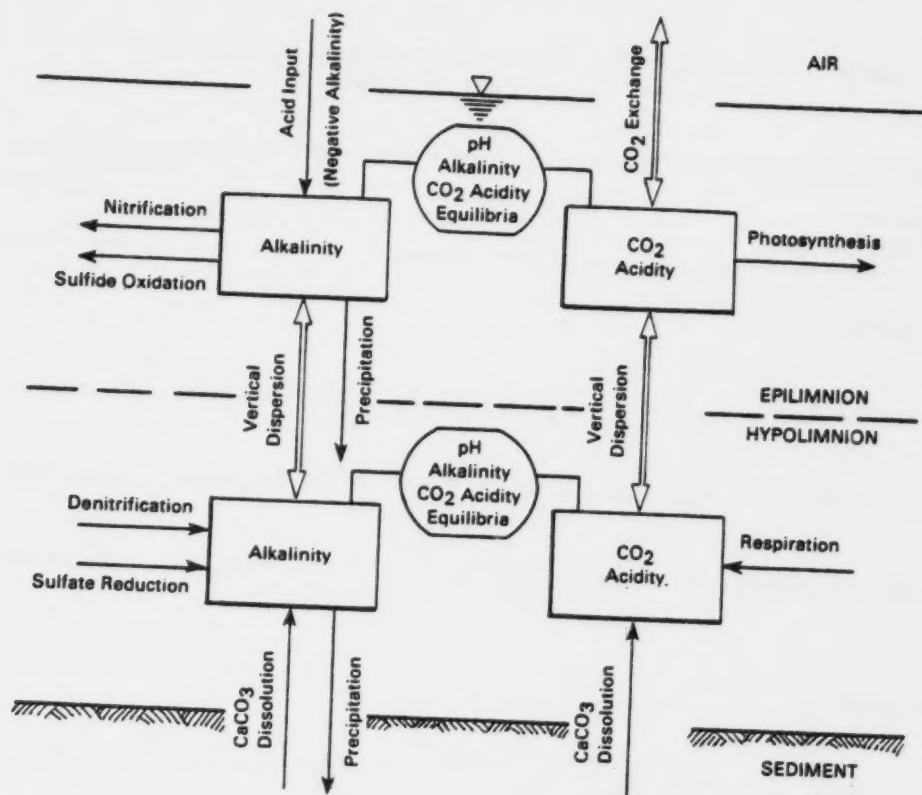
Processes (1)	Reaction (2)	pH (3)	Effect on Alkalinity (4)
In Epilimnion:			
Photosynthesis	$6 \text{ CO}_2 + 6 \text{ H}_2\text{O} \rightarrow \text{C}_6\text{H}_{12}\text{O}_6 + 6 \text{ O}_2$	increase	no change
Nitrification	$\text{NH}_4^+ + 2 \text{ O}_2 \rightarrow \text{NO}_3^- + \text{H}_2\text{O} + 2 \text{ H}^+$	decrease	decrease
Sulfide Oxidation	$\text{HS}^- + 2 \text{ O}_2 \rightarrow \text{SO}_4^{2-} + \text{H}^+$	decrease	decrease
In Hypolimnion:			
Respiration	$\text{C}_6\text{H}_{12}\text{O}_6 + 6 \text{ O}_2 \rightarrow 6 \text{ CO}_2 + 6 \text{ H}_2\text{O}$	decrease	no change
Denitrification	$5 \text{ C}_6\text{H}_{12}\text{O}_6 + 24 \text{ NO}_3^- + 24 \text{ H}^+ \rightarrow 30 \text{ CO}_2 + 12 \text{ N}_2 + 42 \text{ H}_2\text{O}$	increase	increase
Sulfate Reduction	$\text{C}_6\text{H}_{12}\text{O}_6 + 3 \text{ SO}_4^{2-} + 3 \text{ H}^+ \rightarrow 6 \text{ CO}_2 + 3 \text{ HS}^- + 6 \text{ H}_2\text{O}$	increase	increase

Model Structure -- Based on the description of the major model mechanisms, the lake acidification model developed is schematically shown in Fig. 1. Mathematical formulation of the model processes and mechanisms is described in next section.

Model Formulation

CO_2 Exchange Across Air-Water Interface -- The formulation of CO_2 exchange at the air-water interface is,

$$\text{CO}_2 \text{ Mass Rate} = K_L ([\text{CO}_2(\text{s})] - [\text{CO}_2]_e) A \quad (1)$$



GOVERNING EQUATIONS:

$$\frac{\partial [\text{Alk}]}{\partial t} - \frac{1}{A} \frac{\partial}{\partial y} \left(EA \frac{\partial [\text{Alk}]}{\partial y} \right) = S_{\text{Alk}}$$

$$\frac{\partial [\text{CO}_2 \text{ Acy}]}{\partial t} - \underbrace{\frac{1}{A} \frac{\partial}{\partial y} \left(EA \frac{\partial [\text{CO}_2 \text{ Acy}]}{\partial y} \right)}_{\text{Physical Transport}} = \underbrace{S_{\text{CT}} - S_{\text{Alk}}}_{\text{Biological, Chemical \& Physical Processes}}$$

pH - ALKALINITY - CO₂ ACIDITY EQUILIBRIA:

$$[\text{H}^+] = \frac{K_1 ([\text{CO}_2 \text{ Acy}] - K_2 - K_w/K_1)}{\text{Alk} + K_1} \quad \text{and}$$

$$[\text{H}^+] = -[\text{Alk}] \quad \text{if } \text{Alk} < 0$$

Figure 1 Kinetics and Equilibria of Lake Acidification Model

where K_L is the CO_2 transfer coefficient across the lake surface, $[\text{CO}_2(s)]$ is the saturation concentration of dissolved CO_2 in natural waters, $[\text{CO}_2]_e$ is the average concentration of dissolved but unionized CO_2 in the epilimnion, and A is the air-water interface area. In Eq. 1, $[\text{CO}_2]_e$ can be replaced by $[\text{CO}_2 \text{ Acy}]_e$, the concentration of CO_2 acidity in the epilimnion without significant error (9,26). However, a more accurate approximation can be achieved by replacing $[\text{CO}_2]_e$ by $[\text{CO}_2 \text{ Acy}]_e$ less $[\text{H}^+]$ for the pH range normally encountered in acidified waters.

In lakes and impoundment where water movement and turbulence are relatively small compared to those encountered in streams and rivers, wind action is the predominant factor in determining the rate of transfer of dissolved oxygen or carbon dioxide. Experimental investigations and analytical studies of this phenomenon can be found in various studies (12,19,33). In this study, the empirical formulae for dissolved oxygen transfer coefficient developed by Banks and Herrera (3) is adopted:

$$K_L = 0.728W^{1/2} - 0.317W + 0.0371W^2 \quad (2)$$

in which K_L = dissolved oxygen transfer coefficient in m/day at 20°C
 W = windspeed over the lake surface in m/sec

The CO_2 transfer coefficient is derived from the following equation based on the molecular weights of CO_2 and O_2 (28):

$$K_L(\text{CO}_2) = K_L(\text{O}_2) (44/32)^{-2/3} = 0.81K_L(\text{O}_2) \quad (3)$$

The temperature correction for the transfer coefficient is expressed as

$$K_L(T) = K_L(20^\circ\text{C}) \theta^{T-20} \quad (4)$$

in which T is temperature in $^\circ\text{C}$ and θ is an empirical constant ranging from 1.005 to 1.030 with $\theta = 1.024$ being commonly used (19).

The CO_2 saturation concentration, $[\text{CO}_2(s)]$ is determined from the following equation (21):

$$[\text{CO}_2(s)] = e^{[2385.73/T - 17,5184 + 0.0151641T]} \quad (5)$$

in which $[\text{CO}_2(s)]$ is in mole/l at temperature T (in K).

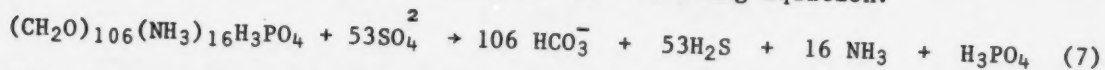
Vertical Diffusion - Fluxes of alkalinity and CO_2 acidity across the epilimnion-hypolimnion interface due to turbulent diffusion and concentration gradients between the two layers are characterized as:

$$\text{Vertical Diffusion Mass Rate} = \frac{\Sigma([\]_e - [\]_h) A}{(L_e + L_h)/2} \quad (6)$$

in which Σ = vertical diffusion coefficient
 $[]_e$ = concentration of alkalinity or CO_2 acidity in the epilimnion
 $[]_h$ = concentration of alkalinity or CO_2 acidity in the hypolimnion
 L_e = average thickness of the epilimnion
 L_h = average thickness of the hypolimnion

The vertical diffusion coefficient is calculated by the flux-gradient method (23) using temperature gradient in the vertical direction on a time variable basis.

Reaction Kinetics -- Most of the biologically mediated processes in Table 1 depend on the kinetics of their reactions. At present, knowledge of the reaction kinetics is not complete, although their chemical equilibrium may be understood. For example, reduction of sulfate by microbial processes produces alkalinity (31). One equivalent of HCO_3^- being produced for each equivalent of SO_4 reduced according to the following equation:



This process may occur in anaerobic hypolimnetic waters, interstitial waters and in anaerobic conditions that occur in winter under ice cover. At what rate, the reaction in Eq. 7 can proceed in a lake requires a significant amount of data to quantify. For most lake, data of this nature do not exist. Therefore, it is extremely difficult to include these processes in the model on an individual basis. A more simplified approach to characterize these processes is required. Thus, in the modeling framework developed, these biologically mediated processes are being grouped collectively in terms of sources and sinks for alkalinity and CO_2 acidity in the epilimnion and hypolimnion. For example, in the epilimnion, photosynthesis decreases CO_2 acidity (but causes no change in alkalinity) and is considered as a sink for CO_2 acidity. On the other hand, respiration which generates CO_2 in the hypolimnion is considered a source for CO_2 acidity. As a first approximation, photosynthesis is considered in the epilimnion and not in the hypolimnion. Likewise, respiration is introduced in the hypolimnion and not in the epilimnion in the present modeling framework. Similarly, the sink for alkalinity in the epilimnion is characterized collectively by nitrification and sulfide oxidation. These two oxidation processes do not cause any change in CO_2 acidity (see Table 1) and therefore not included in the sinks or sources for CO_2 acidity in the epilimnion. In the hypolimnion, denitrification and sulfate reduction are grouped together to characterize the source for alkalinity. Based on the above discussion, the following conceptual expressions are used in the modeling framework to quantify the reaction kinetics in the lake:

Nitrification and sulfide oxidation in the epilimnion			
Decrease of	Kinetic	Alkalinity in	Epilimnetic
alkalinity	= coefficient (μ_1) *	epilimnion	* volume
(mass/time)	(/time)	(mass/volume)	(volume)

Photosynthesis in the epilimnion

$$\text{Reduction of } \text{CO}_2 \text{ acidity} = \text{Kinetic coefficient } (\mu_2) * \text{CO}_2 \text{ acidity in epilimnion} * \text{Epilimnetic volume}$$

$$\text{Denitrification and sulfate reduction in the hypolimnion} \\ \text{Increase of } \text{alkalinity} = \text{Kinetic coefficient } (\mu_3) * \text{Alkalinity in hypolimnion} * \text{Hypolimnetic volume}$$

$$\text{Respiration in the hypolimnion} \\ \text{Increase of } \text{CO}_2 \text{ acidity} = \text{Kinetic coefficient } (\mu_4) * \text{CO}_2 \text{ acidity in hypolimnion} * \text{Hypolimnetic volume}$$

The above empirical approach which approximates the reaction kinetics using a first-order kinetics has been reported in lake eutrophication modeling studies (23,29). The kinetic coefficients μ_1 to μ_4 defined in this formulation will be estimated in the next section.

Model Equations - The governing equations in the modeling framework are presented as follows:

a. Total alkalinity in the epilimnion

$$V_1 \frac{d[\text{Alk}]_1}{dt} = \frac{\xi A_{1,2}}{L_{1,2}} ([\text{Alk}]_2 - [\text{Alk}]_1) - \mu_1 [\text{Alk}]_1 V_1 + D_{\text{Alk}} - Q_{\text{out}} [\text{Alk}]_1 + Q_{1,2} (\alpha [\text{Alk}]_1 + \beta [\text{Alk}]_2) \quad (8)$$

eddy diffusion reaction loading vertical advection

b. CO_2 acidity in the epilimnion

$$V_1 \frac{d[\text{CO}_2 \text{Acy}]_1}{dt} = \frac{\xi A_{1,2}}{L_{1,2}} ([\text{CO}_2 \text{Acy}]_2 - [\text{CO}_2 \text{Acy}]_1) + K_L A ([\text{CO}_2]_{(s)} - [\text{CO}_2 \text{Acy}]_1 + [\text{H}^+]_1) - \mu_2 [\text{CO}_2 \text{Acy}]_1 V_1 + D_{\text{CO}_2} - Q_{\text{out}} [\text{CO}_2 \text{Acy}]_1 + Q_{1,2} (\alpha [\text{CO}_2 \text{Acy}]_1 + \beta [\text{CO}_2 \text{Acy}]_2) \quad (9)$$

eddy diffusion CO_2 exchange of air-water exchange reaction loading outflow vertical advection

c. Total Alkalinity in the hypolimnion

$$V_2 \frac{d[\text{Alk}]_2}{dt} = \frac{\xi A_{1,2}}{L_{1,2}} ([\text{Alk}]_1 - [\text{Alk}]_2) + \mu_3 [\text{Alk}]_2 V_2 - Q_{1,2} (\alpha [\text{Alk}]_1 + \beta [\text{Alk}]_2) + W_{\text{Alk}} \quad (10)$$

eddy diffusion reaction vertical advection ground water input

d. CO_2 acidity in the hypolimnion

$$V_2 \frac{d[\text{CO}_2 \text{Acy}]_2}{dt} = \frac{\xi A_{1,2}}{L_{1,2}} ([\text{CO}_2 \text{Acy}]_1 - [\text{CO}_2 \text{Acy}]_2) + \mu_4 [\text{CO}_2 \text{Acy}]_2 V_2 - Q_{1,2} (\alpha [\text{CO}_2 \text{Acy}]_1 + \beta [\text{CO}_2 \text{Acy}]_2) + W_{\text{CO}_2} \quad (11)$$

eddy diffusion reaction vertical advection ground water input

The notations in Eqs. 8 to 10 are listed in Appendix II.

The developed working equations compute the alkalinity and CO_2 acidity concentrations in the epilimnion and hypolimnion on a time-variable basis. In these calculations, alkalinity and CO_2 acidity can be either negative or positive, depending on the relative concentrations of the various ions. Negative values of alkalinity are numerically equivalent to "mineral acidity" in standard chemical definitions.

Once the alkalinity and CO_2 acidity levels in each layer are calculated, the hydrogen ion concentration is then determined from the $\text{CO}_2/\text{HCO}_3^-/\text{CO}_3^{2-}$ equilibria in the following fashion:

1. $[\text{H}^+] = \frac{K_1 ([\text{CO}_2\text{Acy}] - K_2 - K_w/K_1)}{[\text{Alk}] + K_1}$ if $[\text{Alk}] > 2 \times 10^{-5}$ mole/l
2. $[\text{H}^+] = 3.2 \times 10^{-5} - [\text{Alk}]$ if $-2 \times 10^{-5} < [\text{Alk}] < 2 \times 10^{-5}$ mole/l
3. $[\text{H}^+] = -[\text{Alk}]$ if $[\text{Alk}] < -2 \times 10^{-5}$ mole/l

where $[\text{H}^+]$ = hydrogen ion concentration in mole/l

$[\text{Alk}]$ = total alkalinity concentration in mole/l

$[\text{CO}_2\text{Acy}]$ = CO_2 acidity concentration in mole/l

K_1 = first dissociation constant of carbonate equilibrium (temperature dependent)

K_2 = second dissociation constant of carbonate equilibrium (temperature dependent)

K_w = dissociation constant of water (temperature dependent)

Use of the WASP to Implement Proposed Modeling Framework

In many of the water quality modeling programs currently available, the basic variables and interactions are specified, and it is generally not easy to change the number of variables, or more importantly, the complexity of the interactions. As a result, effort has also been expended to select computer software packages that are more general, open-ended, and depend on user specified interactions for specific problems. The most suitable computer program for this study is the Water Quality Analysis Simulation Program (WASP) originally developed by Hydrosience, Inc. and recently documented for U.S. EPA (10).

Designed to serve as a general purpose code, WASP is very general in nature. The program is flexible enough to provide the modeler with the freedom to describe the kinetic processes and the inputs to these processes, as well as the transport processes and the geophysical morphology or setting that go into the framework of the model. The kinetic and/or transport structures are not "hard wired" in WASP (i.e., the equations are not fixed and thus imbedded in the code). While the lake hydrological transport processes such as inflow, outflow, advective and diffusion flows can be readily incorporated into WASP, a kinetic subroutine is required to implement the source and sink terms of alkalinity and CO_2 acidity in the model.

Model Application

The developed Lake Acidification Model (LAM) has been applied to the Bickford Reservoir in central Massachusetts to test the validity of the general modeling framework. A limited amount of data collected from July to December 1981 (13) was used in deriving the model coefficients.

Bickford Reservoir -- The Bickford Reservoir (Fig. 2), located in the towns of Hubbardston and Princeton, was formed by expanding the original Bickford Pond following the construction of a dam and auxiliary dike. Hydrologically, the Bickford watershed includes an upstream reservoir, Mare Meadow Reservoir, and its drainage area. However, the Bickford Reservoir serves as a backup for the Mare Meadow Reservoir which overflows through an emergency spillway into the Bickford only a few times a year. In addition to the Mare Meadow spillway, two other streams drain the watershed. One of them is the West Wachusettts Brook which contributes over 50% of the inflow to the Bickford Reservoir on an annual basis.

Hydrology and Mass Transport -- The first step in model application to the Bickford Reservoir is to determine the lake hydrology and mass transport. That is, the modeling analysis requires the input of various time-variable inflows, outflows, and exchange flows between the two layers. Thus, the monthly average flows in the West Wachusettts Brook, Mare Meadows Reservoir outflow (only once in July 1981), an unnamed brook, ground water inflow to the hypolimnion, and reservoir outflow at the Bickford spillway were incorporated into the model (Fig. 3). Direct precipitation and evaporation, although a minor portion of the total flow, were also included in the hydraulic budget but are not shown in Fig. 3. The seasonally variable flows reached their minimum levels during the months of August and September, and rose sharply in October 1981.

Mass transport between the epilimnion and hypolimnion in the Bickford Reservoir was characterized by the advective flow from the hypolimnion to the epilimnion as well as by the diffusion flow. Figure 3b shows the advective flow and vertical diffusion coefficient. The vertical diffusion coefficient which is calculated using temperature data ranges from below $1 \text{ cm}^2/\text{sec}$ during the summer stratification months to $9.5 \text{ cm}^2/\text{sec}$ in November and December.

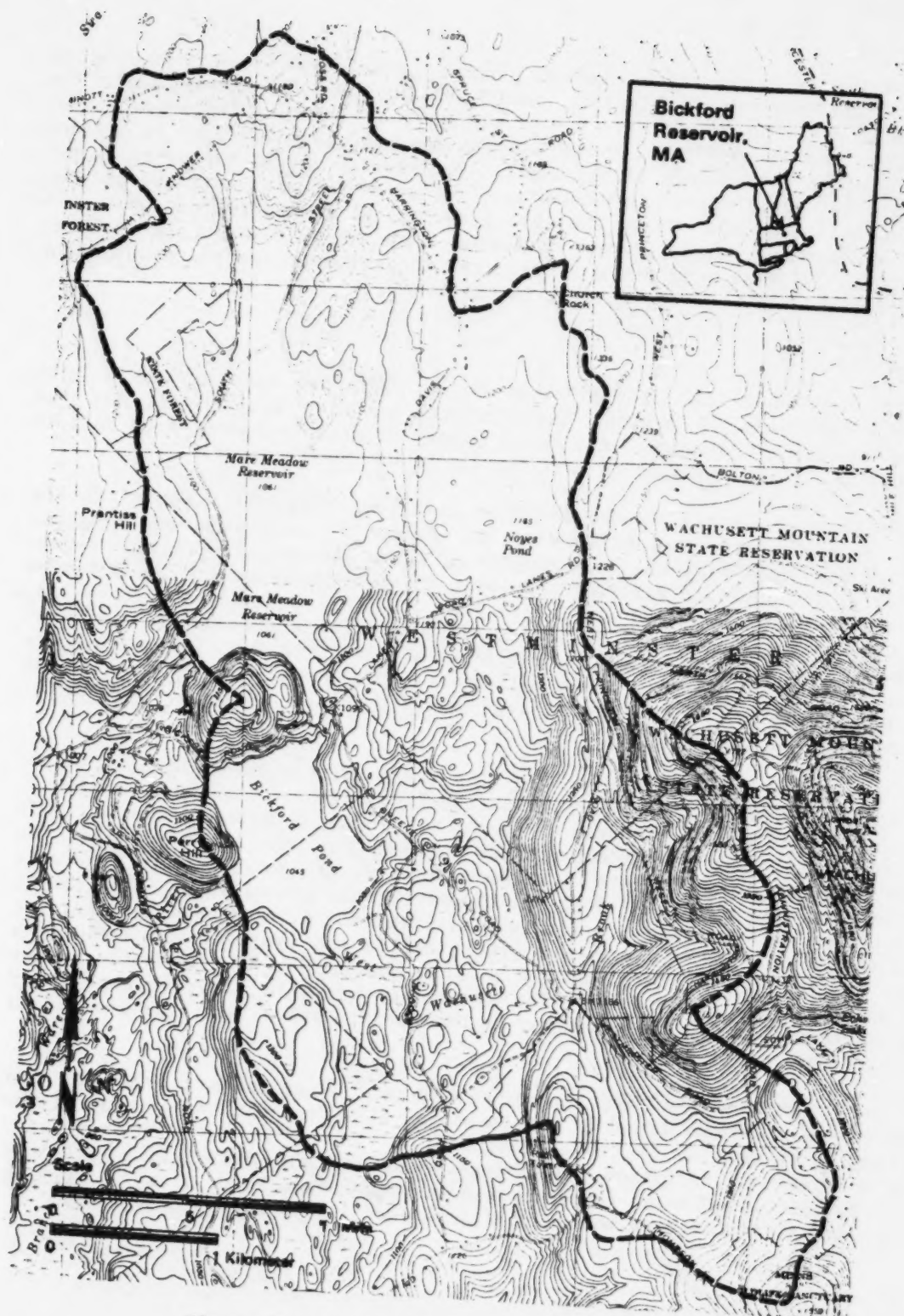


Figure 2 Bickford Reservoir and Watershed

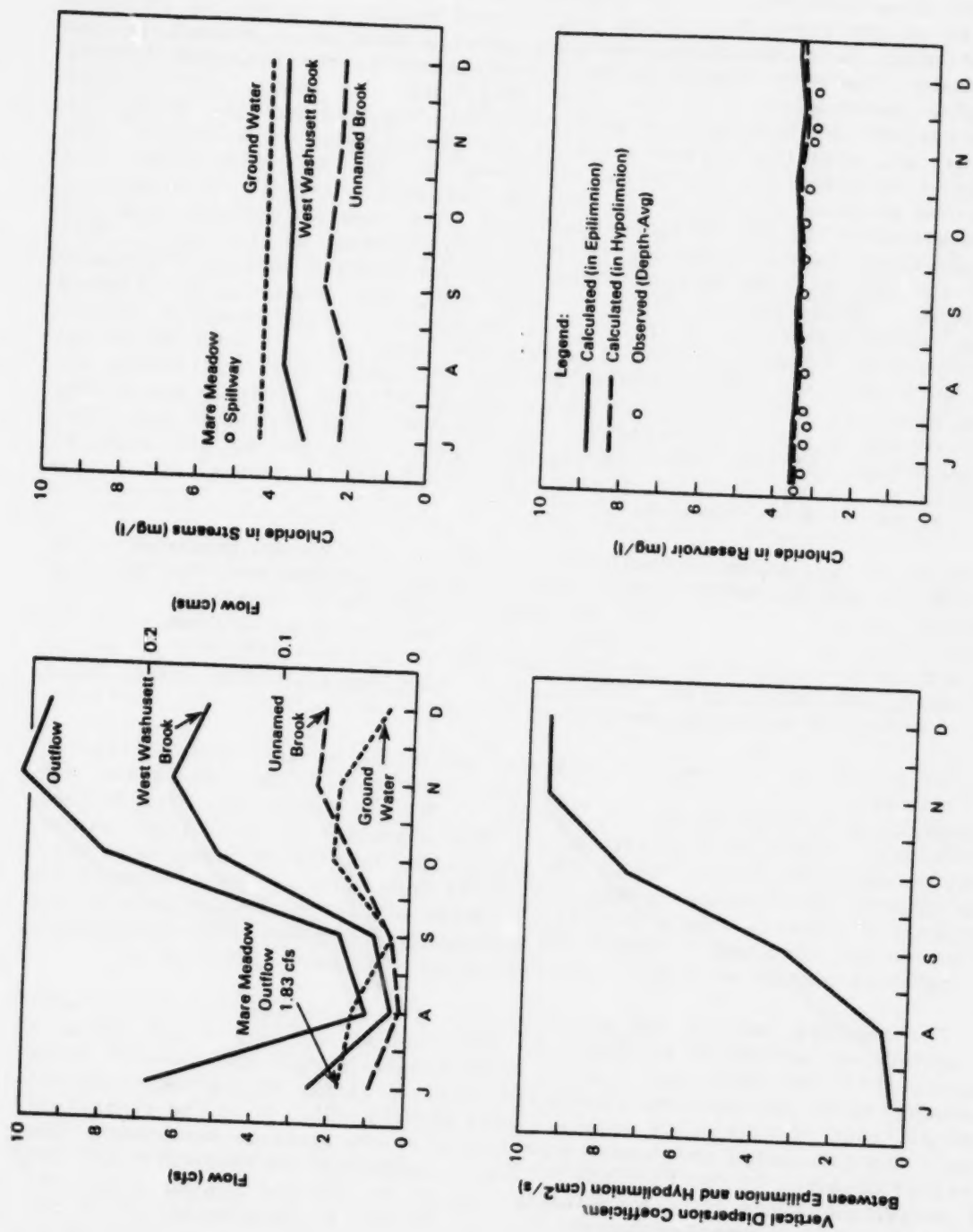


Figure 3 Hydrologic Characteristics of the Bickford Reservoir, Massachusetts, 1981

The above independent derivation of the hydraulic budget and mass transport can be confirmed by applying the derived values to model the chloride concentration (a conservative substance usually used as a tracer) in the impoundment. As such, the chloride concentrations in the reservoir inflows were also incorporated into the model (Fig. 3c). As shown, the chloride concentrations in these inflows were relatively constant and low during the study period, with the highest level in the ground water at 4.2 mg/l. The calculated chloride concentrations are shown in Fig. 3d. Also shown, for comparison purpose, in Figure 3d are the measured chloride concentrations (depth averaged) in the Bickford Reservoir. The results indicate that the chloride level in the reservoir is relatively constant at 3-4 mg/l. This relatively low chloride level is associated with limited influence by man's activities. Being located in an isolated area, the reservoir has limited access, which appears to be the logical explanation of low chloride levels. The calculated values show that the chloride level is very uniform in the lake, such that the concentrations in the epilimnion and hypolimnion are almost identical. The reason for such a steady chloride level is due to the steady concentration of chloride in the reservoir inflows. Also note that the calculated chloride level in the epilimnion is close to the chloride concentration in the West Wachusett Brook which is the predominant inflow (see Fig. 3a) to the epilimnion.

Seasonal Variations of Alkalinity and pH in the Bickford Reservoir -- In addition to the hydraulic budget and transport pattern for the Bickford Reservoir, direct precipitation in terms of alkalinity (negative) and CO_2 acidity concentrations in the tributary flows and ground water flows were also incorporated to analyze the seasonal variations in alkalinity and pH in the epilimnion and hypolimnion. These input data were derived from field measurements (13) and are summarized in Table 2.

The model application also required assigning values to four kinetic coefficients μ_1 , μ_2 , μ_3 , and μ_4 which characterized the generation and consumption of alkalinity and CO_2 acidity in the epilimnion and hypolimnion, respectively. Under this situation, the best means to determine their values was by model calibration. Due to the significant uncertainty associated with these kinetic coefficients, a number of model runs were conducted to match the calculated alkalinity levels with the measured alkalinity levels in both the epilimnion and hypolimnion, while the previously determined hydraulic budget and transport pattern were held constant throughout the calibration.

The modeling results are shown in Fig. 4 and the kinetic coefficients calibrated are presented in Table 3. Examination of Fig. 4 and Table 3 suggests that the calibrated kinetic coefficients produce the results which reasonably match the measured alkalinity levels in terms of the magnitude and seasonal trend of alkalinity levels. Also shown in Fig. 4 is the comparison between the calculated and measured pH levels in the Bickford Reservoir. The pH values measured in the Bickford spillway (outlet of the Reservoir) are used for comparison since they approximate the pH in the epilimnion under the completely mixed assumption for the surface layer. No data on the pH in the hypolimnion are available and, therefore, only the calculated pH values in the hypolimnion are shown in Fig. 4.

Table 2. Model Input for Alkalinity and CO₂ Acidity Analysis of Bickford Reservoir, 1981

Input Variables	July	Aug.	Sept.	Oct.	Nov.	Dec.
Direct Disposition (lb/day) Alkalinity as CaCO ₃	-24.04	-6.45	-19.74	-24.83	-13.10	-24.83
Watershed Contribution of Alkalinity (lb/day as CaCO ₃)	21.94	5.84	8.45	14.88	-21.66	-15.86
Temperature (°C)						
Epilimnion	24.0	23.2	19.0	11.6	8.0	6.0
Hypolimnion	15.0	17.5	15.5	11.5	8.0	6.0
Windspeed (m/sec)	6.0	6.0	7.0	10.2	10.2	8.0

Table 3. Seasonal Variable Biological Kinetic Coefficients for Bickford Reservoir, 1981

Kinetic Coefficient (day ⁻¹)	July	Aug.	Sept.	Oct.	Nov.	Dec.
Alkalinity Consumption in Epilimnion, μ_1	-0.05	-0.24	-0.88	-0.42	-0.28	-0.25
CO ₂ Acidity Consumption in Epilimnion, μ_2	-0.10	-0.15	-0.05	-0.03	-0.05	-0.05
Alkalinity Generation in Hypolimnion, μ_3	0.13	0.38	1.18	1.00	0.25	0.15
CO ₂ Acidity Generation in Epilimnion, μ_4	0.22	0.42	0.72	0.50	0.10	0.05

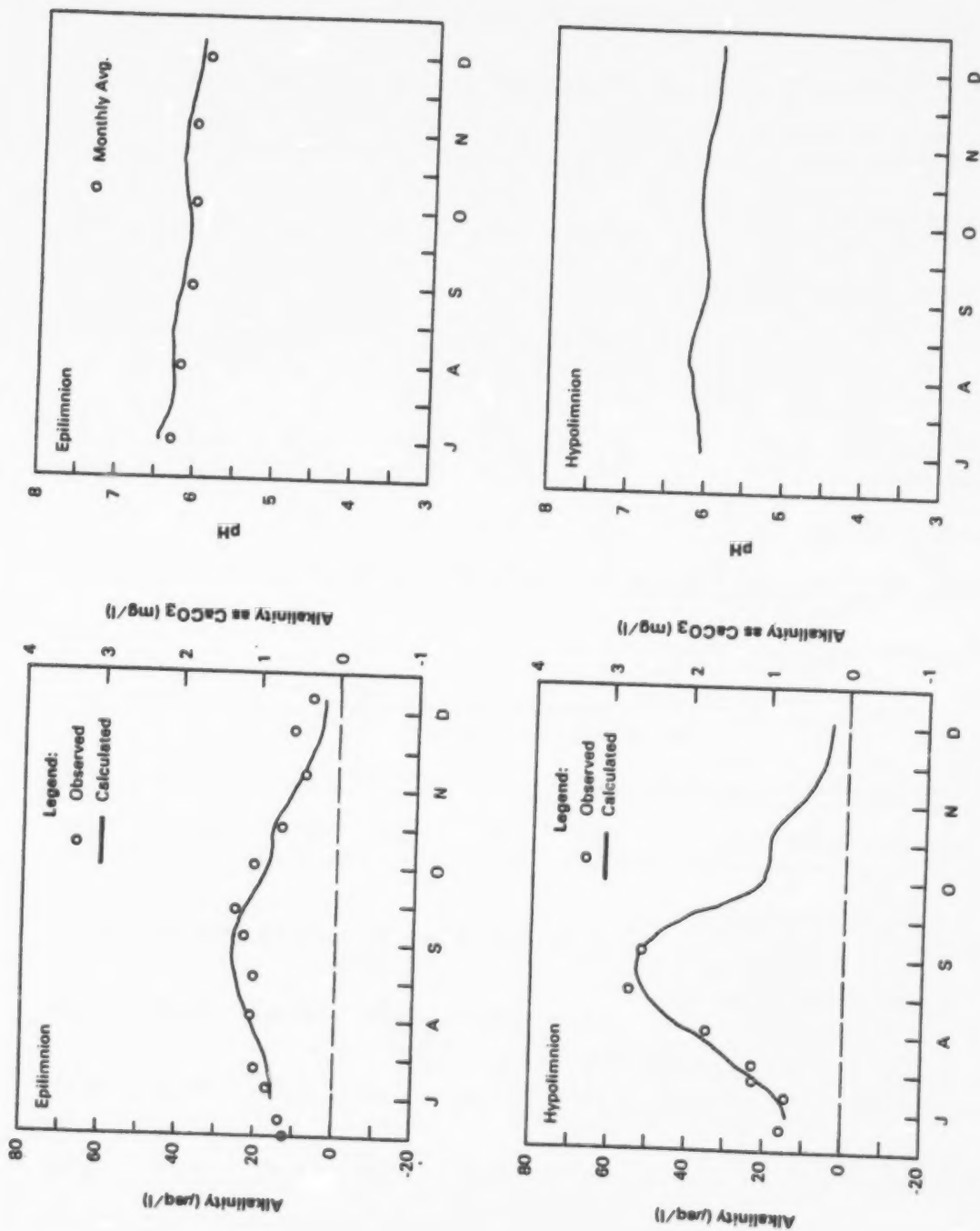


Figure 4 Alkalinity and pH in the Bickford Reservoir, Massachusetts, 1981

CO₂ acidity (or inorganic carbon) was not measured in the Bickford Reservoir (13). Thus, no attempt was made to check the model results for calculated CO₂ acidity. However, since pH is dependent on lake alkalinity and CO₂ acidity in natural waters, favorable comparison between calculated and measured pH levels (Fig. 4) has provided some degree of confidence in CO₂ acidity calculations for the Bickford Reservoir. In addition, the calculated CO₂ acidity levels are all within the reasonable ranges which are reported in the literature.

Discussions of Results

The results in Fig. 4 also provide a good understanding of the alkalinity in the Bickford Reservoir. Beginning in July 1981, alkalinity levels in the epilimnion and the hypolimnion were relatively equal. As the summer stratification progressed through August and September, alkalinity concentration in the hypolimnion increased sharply due to biologically mediated reactions (see Fig. 4 and μ_3 in Table 3). Eshleman (13) reported significant denitrification in the Bickford Reservoir. At the same time, alkalinity concentrations in the epilimnion remained stable and were much lower than those in the hypolimnion. That is because stratification is limited supply of alkalinity from the hypolimnion to the epilimnion. As the lake stratification became less pronounced in October, possibly due to wind-induced mixing in the reservoir, alkalinity levels in the epilimnion increased slightly and alkalinity levels in the hypolimnion decreased. As stream flows increased (see Fig. 3) to further dilute the alkalinity levels in the reservoir, alkalinity levels in the epilimnion and hypolimnion became more uniform and reduced to much lower values.

Examination of the kinetic coefficients in Table 3 indicates that the maximum values of μ_1 , μ_3 , and μ_4 occurred in September when the biological activities were most significant during the year. However, there is no clear trend in the temporal behavior of μ_2 from this modeling calibration exercise. In addition, the magnitudes of μ_2 are small, much smaller than those of μ_1 , μ_3 , and μ_4 . It appears that the CO₂ consumption rate (possibly due to photosynthesis) in the epilimnion was low (i.e., very insignificant algal production). Thus, the μ_2 values shown in Table 3 are nothing more than the system fluctuations about a relatively small mean value.

Summary and Conclusions

A modeling framework for lake acidification assessment is developed by incorporating two model variables -- alkalinity and CO₂ acidity in a two-layer (epilimnion and hypolimnion) fashion. Biologically mediated processes in the epilimnion and hypolimnion are incorporated with physical processes such as CO₂ exchange at air-water interface and vertical diffusion between the epilimnion and hypolimnion. The CO₂/HCO₃⁻/CO₃²⁻ equilibria are also included to determine pH levels based on the calculated alkalinity and CO₂ acidity concentrations. In addition, external inputs from the atmosphere and the ground water system are also included.

The Water Quality Analysis and Simulation Program (WASP) is used as the general computer code to implement the above modeling framework. A kinetic subroutine has been specifically written to formulate the kinetic processes for alkalinity and CO₂ acidity. The formulation of the individual biologically mediated processes is not explicit in the model at the present time due to the limited understanding of the processes. Instead, these processes are grouped together as the sources and sinks of alkalinity and carbon dioxide acidity via parameterization. Designed to be general in nature, WASP is flexible enough to provide the freedom to incorporate the kinetic processes and the inputs to these processes, as well as the transport processes and the geophysical morphology or setting that go into the framework of the model. This model has been found running very efficiently on both main frame and mini-computer systems.

The developed modeling framework has been applied to the Bickford Reservoir in Massachusetts. The modeling results indicate that the seasonal variations of alkalinity in the epilimnion and hypolimnion of the Bickford Reservoir are reproduced for the 1981 conditions. The model results explain the generation of alkalinity in the hypolimnion of the reservoir. The general applicability of the modeling framework to acidified lakes can be expanded by applying the framework to other lakes.

Appendix I - References

1. Almer, B.W., Dickson, W., Ekstrom, C., and Hornstrom, E., "Sulfur Pollution and the Aquatic Ecosystem," Sulfur in the Environment, Part II: Ecological Impacts, J.O. Nriagu, ed., Wiley-Interscience, New York, NY, 1979, pp. 271-311.
2. Altwicker, E.R. and Johannes, A.H., "Wet and Dry Deposition into Adirondack Watersheds," The Integrated Lake-Watershed Acidification Study: Proceedings of the ILWAS Annual Review Conference, Electric Power Research Institute EA-2827, 1983.
3. Banks, R.B. and Herrera, F.F., "Effect of Wind and Rain on Surface Reaeration," ASCE Journal of Environmental Engineering Division, Vol. 103, No. EE3, June 1977, pp. 489-504.
4. Brosset, C., "Factors Influencing pH in Lake Water," Swedish Water and Air Pollution Research Institute Report B443, 1978, 9p.
5. Brosset, C., "The Acid-Base Balance in Lake Water," Swedish Water and Air Pollution Research Institute Report B540, 1980, 21p.
6. Brown, W.E., "Technique to Assess the Impacts of Acid Rain on Surface Water Supplies, Proceedings of American Society of Civil Engineers 1982 National Conference on Environmental Engineering, Minneapolis, Minnesota.
7. Chadderton, R.A. and Shane, R.M., "A Model of the Behavior of pH Determining Parameters in Impoundments," Water Resources Bulletin, Vol. 14, No. 6, 1978, pp. 1357-1373.
8. Chen, C.W., Gherini, S., and Goldstein, R., "Modeling the Lake Acidification Process," Paper presented at the Lake Acidification Workshop, Sept. 4-7, 1978, Central Electricity Generating Board, England.
9. Di Toro, D.M., "Combining Chemical Equilibrium and Phytoplankton Models-- A General Methodology," Modeling Biochemical Processes in Ecosystems, R.P. Canale, ed., Ann Arbor Science Publishers, Inc., 1976, pp. 233-255.
10. Di Toro, D.M., J.J. Fitzpatrick, and R.V. Thomann, "Water Quality Analysis Simulation Program (WASP) and Model Verification Program (MVP) Documentation," EPA Report 1980.
11. Edzwald, J.K. and De Pinto, J.V., "Recovery of Adirondack Lakes with Fly Ash Treatment," Final Report No. RC-A-75-4, Engineering Foundation Grant, 1978.
12. Eloubaidy, A.F. and Plate, E.J., "Wind Shear-Turbulence Diffusion and the Reaeration Coefficient," ASCE Journal of Hydraulics Division, Vol. 98, No. HY1, Jan. 1972, pp. 153-170.

13. Eshleman, K., "A Biogeochemical Mass Balance Approach to Reservoir Acidification by Atmospheric Deposition," MS Thesis, MIT Department of Civil Engineering, 1982, 117p.
14. Hendrey, G.R., Galloway, J.N., and Schofield, C.L., "Temporal and Spatial Trends in the Chemistry of Acidified Lakes under Ice Cover," Electric Power Research Institute Research Project 1109-5 Interim Report, May 1981, pp.7-1 to 7-5.
15. Hendrey, G.R. and Kaplan, E., "Identification of Freshwaters Susceptible to Acidification," Brookhaven National Laboratory, Upton, NY, BNL-31000, CONF-820240-1, 7p.
16. Henriksen, A., "Acidification of Freshwaters: A Simple Approach for Identification and Quantification," Nature, Vol. 278, 1979, pp.542-545.
17. Henriksen, A., "Acidification of Freshwaters--A Large Scale Titration," Ecological Impact of Acid Precipitation, Proceedings of An International Conference, Sandefjord, Norway, March 11-18, 1980, pp.68-74.
18. Henriksen, A. and Wright, R.F., "Effect of Acid Precipitation on a Small Acid Lake in Southern Norway," Nordic Hydrology, Vol. 8, 1977, pp.1-10.
19. Holley, E.R., "Oxygen Transfer at the Air-Water Interface," In Transport Processes in Lakes and Oceans, Gibbs, R.J., ed., Plenum Press, New York, 1977.
20. Jassby, A. and Powell, T., "Vertical Patterns of Eddy Diffusion During Stratification in Castle Lake, California," Limnology and Oceanography, Vol. 20, No. 3, pp.530-543.
21. Kelly, M.G., Church, M.R., and Hornberger, G.M., "A Solution of the Inorganic Carbon Mass Balance Equation and its Relation to Algal Growth Rates," Water Resources Research, Vol. 10, No. 3, June 1974, pp. 493-497.
22. Kramer, J. and Tessier, A., "Acidification of Aquatic Systems: A Critique of Chemical Approaches," Environmental Science & Technology, Vol. 16, No. 11, 1981, pp.606A-615A.
23. Lung, W.S. and Canale, R.P., "Projections of Phosphorus Levels in White Lake, ASCE Journal of Environmental Engineering Division, Vol. 103, No. EE4, pp.663-670.
24. Lung, W.S., "Development of a Lake Acidification Model," Paper presented at the Technical Symposium on Acid Precipitation Transport and Transformation Phenomena, University of Vermont, Burlington, VT. September 22, 1983.

25. Schindler, D.W., Wageman, R., Cook, R.B., Ruszczynski, I., and Prokopowich, J., "Experimental Acidification of Lake 223, Experimental Lake Area: Background Data and First Three Years of Acidification," Canadian Journal Fishery and Aquatic Science, Vol. 37, No. 3, 1980, pp.342-354.
26. Schnoor, J.L., Carmichael, G.R. and Van Schepen, F.W., "An Integrated Approach to Acid Rainfall Assessments," Energy and Environmental Chemistry, Vol 2, Acid Rain, L. H. Keith, ed., Ann Arbor Science Publishers, Inc., 1982, pp.225-243.
27. Schofield, C.L., "Effects of Acid Rain on Lakes," Acid Rain, Gunnerson, C.G. and Willard, B.E., eds., ASCE, New York, NY, 1979, pp.55-69.
28. Shen, T.T., "Hazardous Air Emissions from Industrial Waste Treatment Facilities," Proceedings of 14th Mid-Atlantic Industrial Waste Conference, College Park, Maryland, June 27-29, 1982.
29. Stumm, W. and Leckie, J.O., "Phosphate Exchange with Sediments; its Role in the Productivity of Surface Waters," Paper presented at the 5th International Water Pollution Research Conference, July-August 1970.
30. Stumm, W. and Morgan, J.J., Aquatic Chemistry, John Wiley and Sons, New York, NY, 1981.
31. Weber, W. J., Jr. and Stumm, W., "Mechanisms of Hydrogen Ion Buffering in Natural Waters," Journal American Water Works Association, Vol. 55, No. 12, pp.1553-1578.
32. Yeasted, J.C. and Shane, R., "pH Profiles in a River Systems with Multiple Acid Loads," Journal Water Pollution Control Federation, Vol. 48, No. 1, 1976, pp.91-106.
33. Yu, S.L., "Atmospheric Reaeration in a Lake." Department of Civil and Environmental Engineering Report, Rutgers University, New Brunswick, NJ, August 1977, 50p.

Appendix II - Notation

The following symbols are used in this paper:

- A = lake surface area;
- $A_{1,2}$ = interfacial area between epilimnion and hypolimnion;
- $[Alk]_1, [Alk]_2$ = alkalinity concentrations in the epilimnion and hypolimnion, respectively;

- $[\text{CO}_2\text{Acy}]_1, [\text{CO}_2\text{Acy}]_2$ = CO_2 acidity concentrations in the epilimnion and hypolimnion, respectively;
- $[\text{CO}_2(s)]$ = saturated concentration of CO_2 ;
- $[\text{CO}_2]_e$ = CO_2 concentration in the epilimnion;
- $D_{\text{Alk}}, D_{\text{CO}_2}$ = loading rates of alkalinity and CO_2 acidity from direct deposition and tributary input to the epilimnion;
- $[\text{H}^+]$ = concentration of hydronium ion;
- K_1, K_2 = first and second dissociation constants of carbonate equilibria;
- K_w = dissociation constant of water;
- $K_L(\text{CO}_2)$ = reaeration coefficient for CO_2 ;
- $K_L(\text{O}_2)$ = reaeration coefficient for O_2 ;
- L_e = average depth of the epilimnion;
- L_h = average depth of the hypolimnion;
- $L_{1,2}$ = average of the epilimnion depth and hypolimnion depth;
- $Q_{1,2}$ = advective flow between the epilimnion and hypolimnion;
- Q_{out} = outflow from the epilimnion;
- T = temperature;
- V_1, V_2 = volumes of the epilimnion and hypolimnion respectively;
- W = windspeed;
- $W_{\text{Alk}}, W_{\text{CO}_2}$ = ground water input of alkalinity and CO_2 acidity respectively;
- θ = temperature correction constant for reaeration coefficient;
- $\mu_1, \mu_2, \mu_3, \mu_4$ = first-order kinetic coefficients for alkalinity and CO_2 acidity in the epilimnion and hypolimnion, respectively;
- α, β = weighting factors to determine the concentrations at the epilimnion-hypolimnion interface; and
- ξ = vertical diffusion coefficient in the water column.

SENSITIVITY ANALYSIS OF SWMM PREDICTIONS
ON WATER QUALITY IN THE DETROIT RIVER

by

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Abstract

The Fourier Amplitude Sensitivity Test (FAST) has been applied to the Detroit River Plume Model employed in the Detroit 201 study. The plume model superimposes all SWMM CSO sources, the Detroit Wastewater Treatment Plant Plume and the confluence of the Rouge River to obtain composite predictions of concentrations in the Detroit River. First order error analysis could not be applied to a highly nonlinear case such as this. The partial variances for dissolved oxygen, fecal coliform, and total phosphorus concentrations are calculated as a function of the model input parameters: the turbulent diffusivity, the river velocity and depth and the background concentrations and the SWMM model output waste loadings.

The resulting sensitivity coefficients as a function of position in the River are displayed and the significance of the results are discussed.

Introduction

The Detroit Water and Sewerage Department provides wastewater collection and treatment services over an area encompassing 650 square miles and including 3,200,000 people and over 1500 industrial dischargers. About 62 percent of the service area is served by separated storm and sewage sewers; the rest is served by combined sewers.

Once wastewater reaches the city, flow is generally towards the Detroit or Rouge Rivers where major interceptors divert dry weather flow to a single, large treatment plant near the confluence of the two rivers. Significant rainfall creates runoff in excess of interceptor capacity which, by design, overflows to the Detroit and Rouge Rivers at approximately 80 locations, called combined sewer overflows (CSO's).

In September 1977 a Consent Judgement mandated that the quantity and quality of combined sewer overflows from the City of Detroit be determined.

The objectives of the Section 201 Study that followed were:

1. To determine the quantity and quality of combined sewer overflows to the Detroit and Rouge Rivers.
2. To quantify the impacts of combined sewer overflows on the Rouge River, Detroit Waste Water Treatment Plant, and Detroit River for the purposes of facilities planning.
3. To provide a tool for evaluating the impacts of various CSO control alternatives on the Detroit and Rouge Rivers.

The area modeled included the entire City of Detroit along with some small portions of a few suburban communities which totaled approximately 88,000 acres. The evaluation of potential alternatives was conducted using the USEPA Storm Water Management Model (SWMM), RUNOFF and TRANSPORT blocks, supplemented by QUAL II, RECEIVE II, and models developed in the 201 study to predict the dynamic behavior of the 800MGD treatment facilities and one, two and three dimensional dynamic impacts on receiving streams. The modeling package consisted of five coupled models, including 1) the collection system model, 2) the Rouge River model, 3) the Detroit Wastewater Treatment Plant (DWWTP) model STPSIM2, 4) a Plume model (Detroit River near shore), and 5) the overall Detroit River model (USSMPX).

The details of the overall 201 study and the models employed have been given by Kummler [1982] and references therein, and will not be repeated here. An uncertainty analysis of the modeling package (1) through (4) was conducted by Kummler, et al. [1981] using Monte Carlo techniques. In this work we describe a more formal procedure for uncertainty analysis using the Fourier Amplitude Sensitivity Technique. We focus herein on the Plume Model using SWMM as input.

The Gaussian Plume Model

The basic equation governing the conservation of mass in flowing fluids is the equation of continuity for the time averaged concentration, assuming stationary, homogeneous turbulence, which should be an excellent assumption for extensive reaches of the Detroit River. The continuity equation for species i , then becomes, for flow aligned in the x direction, neglecting dispersion in the axial direction and assuming complete vertical mixing:

$$\frac{\partial c_i}{\partial t} + \frac{\partial c_i}{\partial x} = K_y \frac{\partial^2 c_i}{\partial y^2} \quad (I)$$

If K_y , the lateral diffusion coefficient, is a constant, then there is an exact solution for a continuous point source located at (x_0, y_0) in an unbounded river

$$\bar{c}_i(x, y, p) = \frac{Q_i}{2h\sqrt{K_y \pi x u}} \left\{ \exp\left[-\frac{(y-p)^2}{4K_y x/u}\right] + \exp\left[-\frac{(y+p)^2}{4K_y x/u}\right] \right\} \exp\left(-\frac{kx}{u}\right) \quad (II)$$

as presented by Seinfeld (1975), where $\bar{c}_i$ = concentration of species i at

location (x,y) due to a source of strength Q_i located at (x_o, y_o) ; u = river velocity; h = average river depth; x = distance from the source; y = distance from the shore line; p = effective lateral penetration; and k = rate constant for a first order kinetic decay.

The model employed for the Detroit 201 study was a superposition solution allowing for 48 sources, an arbitrary x,y grid pattern, and coupled first order reactions. The model has been presented by Liang, et al. (1981). The experimental verification of the model was conducted by Liang (1981).

The FAST Method

The method to be briefly described in this section was developed by Shuler and coworkers (see, for example, Cukier, et al. (1973) and (1975)) and reduced to computer code by McRae, et al. (1982) and by Koda, et al. (1979). The version employed by our group was obtained through Norbeck (1983).

In general, modeling requires the solution of either

1. a set of n coupled differential equations

$$\frac{dx}{dt} = f(x,t;k) \quad \text{or}$$

2. a system of algebraic equations

$$F(x;t;k) = 0$$

where x is an n dimensional state vector, and
 k is an m dimensional parameter vector.

The problem we are addressing herein is to understand what effect variations in the input k 's will have on the output x 's at various values of t . Taylor's theorem is often used (neglecting higher order terms) to measure the sensitivity of x to k in accordance with

$$\Delta x_i(t,k) = x_i(t, k+\Delta k) - x_i(t,k) = \sum_{j=1}^m \frac{\partial x_i}{\partial k_j} \Delta k_j + f(\Delta k_j^2) + \dots$$

where

$\frac{\partial x_i}{\partial k_j}$ is called the sensitivity coefficient.

The Taylor Series truncation provides a local sensitivity analysis because it is only applicable to small variations of Δk about its nominal value. It does not assess the effect of simultaneous large variations in all parameters. On the other hand, what is actually desired for most complex modelling, is a global sensitivity analysis which measures the sensi-

tivity of the model's output to the total range of uncertainty of the model. It is usually totally impractical, costly, and misleading to attempt a Taylor Series analysis for a complex numerical model. The FAST method provides a global analysis with reasonable cost.

Definitions

If the probability density function for a given x_i at specific values of the parameters k_m is given by $p(k_1, k_2 \dots k_m)$, then the ensemble mean for x_i is given by:

$$\langle x_i(t) \rangle = \int \dots \int x_i(t; k_1, k_2 \dots k_m) p(k_1, k_2 \dots k_m) dk_1 \dots dk_m$$

and the variance by:

$$\sigma_i^2(t) = \langle x_i(t)^2 \rangle - \langle x_i(t) \rangle^2$$

The m dimensional integral for the ensemble mean $\langle x_i(t) \rangle$ can be converted into a one dimensional form by the transformation

$$k_\ell = G_\ell[\sin(w_\ell s)] \quad \ell = 1, 2 \dots m,$$

where the G_ℓ functions are chosen such that the arc length, ds , is proportional to $p(k_1, k_2 \dots k_m)$ for all ℓ .

It has been shown that the G_ℓ 's can be chosen (Weyl, 1938) such that

$$\bar{x}_i = \langle x_i(t) \rangle \sim \lim_{T \rightarrow \infty} \frac{1}{2T} \int_{-T}^T x_i[k_1(s), k_2(s) \dots k_m(s)] ds$$

The use of integer frequencies causes the k_ℓ 's to be periodic functions in s and the s -space search curve is closed. Then, the Fourier Coefficients:

$$A_{pw_\ell}^{(i)} = \frac{1}{2\pi} \int_{-\pi}^{\pi} x_i[k_1(s) \dots k_m(s)] \cos(pw_\ell s) ds$$

$$B_{pw_\ell}^{(i)} = \frac{1}{2\pi} \int_{-\pi}^{\pi} x_i[k_1(s) \dots k_m(s)] \sin(pw_\ell s) ds$$

$$p = 1, 2 \dots m$$

are a measure of the sensitivity of x_i to the k_i variation.

The ensemble average, $\langle x_i \rangle$, can therefore be expressed in terms of the 0th Fourier coefficients

$$\langle x_i \rangle^2 = A_0^{(i)2} + B_0^{(i)2} = A_0^{(i)2}$$

The frequency variance is

$$\sigma_{wl}^{(i)^2} = 2 \sum_{p=1}^{\infty} [A_{pwl}^{(i)^2} + B_{pwl}^{(i)^2}]$$

Both the frequency variance, $\sigma_{wl}^{(i)^2}$, and the total variance, $\sigma(i)^2$ be calculated. The partial variance is defined as the ratio:

$$S_{wl}^{(i)} = \frac{(\sigma_{wl}^{(i)})^2}{(\sigma(i))^2} = \frac{2}{\sigma_i^2} \left[\sum_{j=1}^{\infty} A_{jwl}^{(i)^2} + B_{jwl}^{(i)^2} \right]$$

The ability to determine which is the most important uncertainty in a complex calculation is unique to the FAST method.

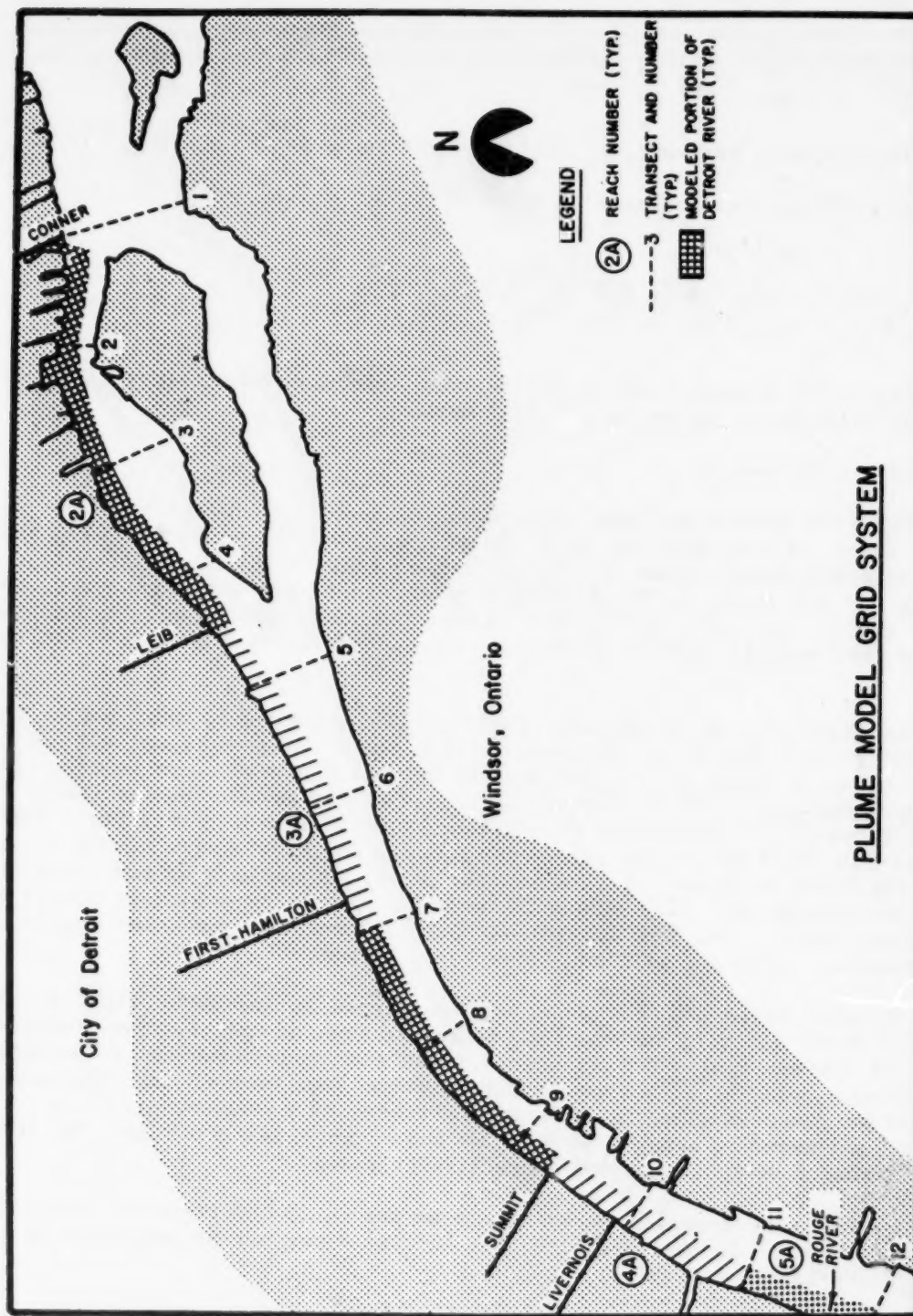
Results and Discussion

The FAST method has been applied to the case of CSO input to the Detroit River in our previous work (El-Sharkawy, et al. (1983)). Therein, we considered single plume discharges and an analysis of the full model allowing for variation in the diffusion coefficient, the river velocity, the river depth, the background concentration and the temperature. Herein, we extend that work to include variations in the SWMM input flux to the Detroit River.

In Figure 1, we illustrate the locations of the CSO stations on the Detroit River. The major stations include three (401, 402, and 403) which enter Conner's Creek before joining the Detroit River and Leib (410). Leib is approximately 16,773 ft downstream from Conner's Creek. We use the plume model to examine concentrations on the USA side of the Detroit River. There are no CSO stations on the Canadian side and Belle Isle effectively separates the River into two portions at this point. Hence only US contributions need be considered. The SWMM model provides the input pollutant fluxes to the plume model. The SWMM results for Detroit have been described previously (Kummler, 1982) and will not be repeated herein.

We will focus in this paper on variations in the SWMM output to the Detroit River and will use the FAST method to examine the relative contribution of various upstream stations to downstream concentrations at selected locations. We will also examine the relative magnitude of the flux uncertainty as compared with the uncertainty caused by turbulent diffusion, velocity and depth. Lastly, we will evaluate the total error and the relative standard deviation in the overall calculation.

We begin by considering outflow from the three CSO stations located at Conner's Creek. In this vicinity, the Detroit River may be described by the nominal variables in Table 1.



PLUME MODEL GRID SYSTEM

FIGURE 1.

Table 1
Nominal Values for the Detroit River

Turbulent Diffusivity, $\bar{K}_y$, ft ² /hr:	2180
Velocity, $\bar{u}$, ft/hr:	7250
Depth, $\bar{h}$, ft:	26

The characteristics of the outfalls at the CSO stations considered in these calculations are given in Table 2.

Table 2
Outflow Concentrations at Each Station

Station No.	BOD mg/l	DO* mg/l	SS mg/l	VSS mg/l	DS mg/l	P mg/l	CAD mg/l	COLI #/100 ml	Waste Flow (CFS)
401	20.69	5.97	80.70	52.70	228.00	0.791	6.712	6.70×10^5	2101.7
402	19.59	5.63	76.40	49.90	201.20	0.724	10.71	1.7×10^6	412.6
403	16.70	5.62	71.50	46.60	200.70	0.590	0.25	5.14×10^5	1033.0
408	53.06	5.88	237.4	154.8	674.7	0.664	0.0048	5.96×10^5	79.6
410	38.36	5.54	139.5	91.0	373.9	1.057	1.1830	2.71×10^6	1454.0
Background Concentration									
C_{oi}	1.0	9.42	1.00	18.00	95.00	0.01	0.00033	19.0	

*The outflow concentration of DO is given as the DO deficit. The background concentration represents the actual concentration.

The relative location of the CSO stations along the US side of the river beginning arbitrarily at the inactive Fox Creek CSO are given in Table 3.

Procedurally, we varied the waste volumetric flow rate which has the effect of varying the mass flux for all of the species simultaneously.

Hence, a single parameter can be used to analyze the uncertainty contribution of the source function for all eight parameters considered in the Detroit 201 study. Herein, we will illustrate results using TP and Coliform.

Table 3
Relative Location of Detroit River CSO Sites
Along the US Side

L	XQ(FT)	YQ(FT)	PQ(FT)	STATION NAME	STAT. NO.
1	0.0	0.0	1.000E+02	FOX CREEK	480
2	6.80000E+03	0.0	1.000E+02	CONNER CK BKWATER GATE	401
3	6.80000E+03	0.0	1.000E+02	CONNER PUMP STATION	402
4	6.80000E+03	0.0	1.000E+02	FREUD PUMP STATION	403
5	1.39240E+04	0.0	1.000E+00	FAIRVIEW PUMP STATION	404
6	1.40180E+04	0.0	1.000E+00	MC CLELLAN	405
7	1.59900E+04	0.0	1.000E+00	FISCHER	406
8	1.66530E+04	0.0	1.000E+00	IROQUOIS	407
9	2.07370E+04	0.0	1.000E+00	HELEN	408
10	2.30470E+04	0.0	1.000E+00	MT. ELLIOTT	409
11	2.35730E+04	0.0	4.000E+01	LEIB	410
12	2.43730E+04	0.0	1.000E+02	ADAIR	411
13	2.60970E+04	0.0	1.000E+00	JOS. CAMPAU	412
14	2.70190E+04	0.0	1.000E+00	CHENE	413
15	2.76220E+04	0.0	1.000E+00	DUBOIS	414
16	2.80610E+04	0.0	1.000E+00	ST. AUBIN	415
17	2.91000E+04	0.0	1.000E+00	ORLEANS	416
18	2.91000E+04	0.0	1.000E+00	ORLEANS RELIEF	417
19	2.95400E+04	0.0	1.000E+00	RIOPELLE	418
20	3.06260E+04	0.0	1.000E+00	RIVARD	419

In Figure 2, we present the station by station partial sensitivity of the total phosphorus (TP) concentrations in the Detroit River at a point 70 ft offshore as a function of downstream distance. This figure shows that the partial sensitivity is a strong function of the mass flux associated with each station. Station 401 has the highest flow rate and the highest TP concentration and therefore resulting concentrations in the river are most sensitive to uncertainties in the mass flux from station 401. In Figure 3, the similar analysis for coliform is presented. Because the fecal coliform concentration from station 403 is only a third of the concentration from station 402, we see that the relative importance of stations 402 and 403 is reversed from that displayed in Figure 2. In both cases illustrated in Figures 1 and 2, only the input flux, Q_i , is varied; hence, the partial sensitivities are independent of longitudinal, or downstream, distance. Examination of equation II and a first order error analysis would lead to the same conclusion. The relative standard deviation, defined as the total standard deviation divided by the nominal calculated concentration is displayed in Figure 4 as a function of longitudinal distance. Again, the relative standard deviation is insensitive to distance, but only because the background concentration of TP in the Detroit River is low. There is a small trend toward lower relative standard deviation downstream as expected.

Next we consider variations in stations 401, 408, and 410. The distance between these stations are given in Table 3.

FIGURE 2.

PARTIAL SENSITIVITY VS. LONGITUDINAL DISTANCE FOR TP AT Y=90.0FT.

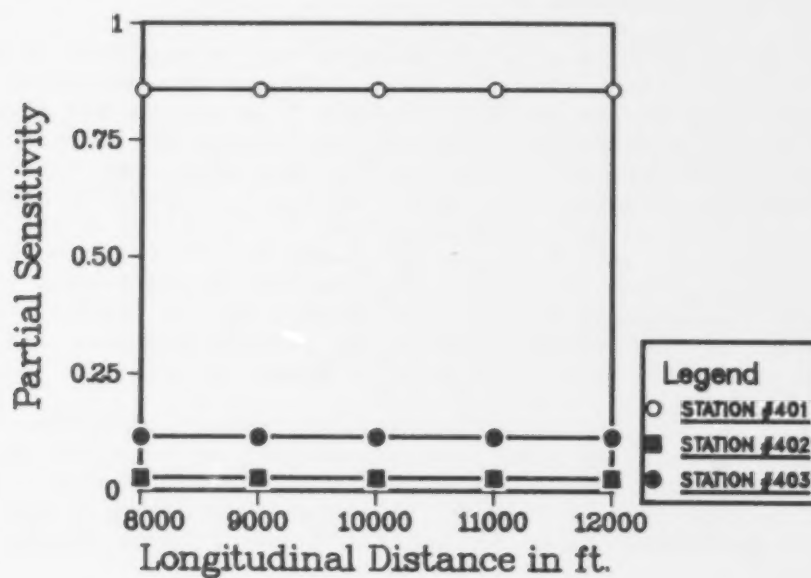
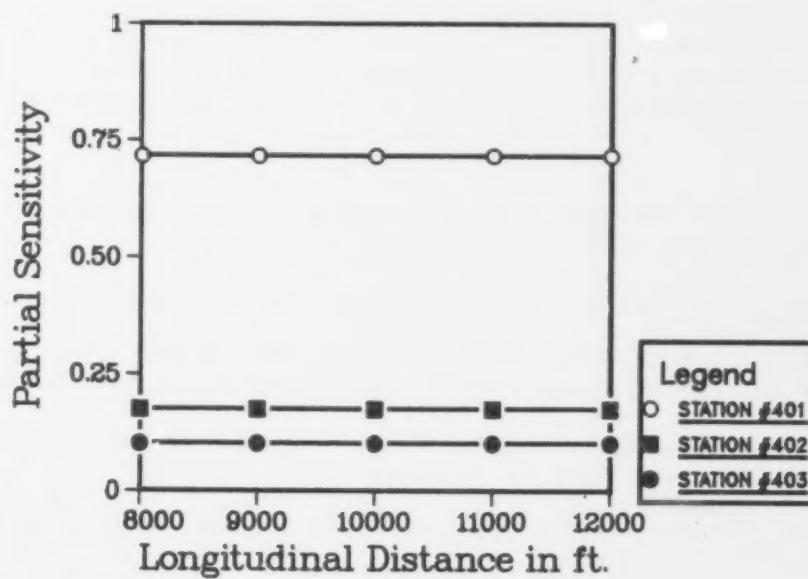


FIGURE 3.

PARTIAL SENSITIVITY VS. LONGITUDINAL DISTANCE FOR COLIFORM AT Y=90.0FT.



In Figure 5, the relative standard deviation is presented as a function of downstream distance. There, we see that the total deviation is high where station 401 contributes (up to 14,000 ft), is zero where the flow has not yet reached during the time selected and increases past 23,000 ft due to the contribution of station 410. The contribution of station 408 is very small and is barely discernable.

Next, we consider a case in which the effect of mass flux is examined together with the effects of $\bar{K}_y$, u , and h . The source strength of station 401 is varied while others are held constant. The results are given in Figure 6. At short distances downstream from station 401 velocity is a major factor in the partial sensitivity, but far downstream, river depth is the most important variable. The effect of source strength as measured by the partial sensitivity due to flow accounts for about 15% of the total error. For the same case, the relative standard deviation is portrayed in Figure 7. The magnitude of the uncertainties has caused roughly a factor of two overall uncertainty in the final result. As the concentration drops downstream so does the relative deviation. As expected when more parameters are allowed to be uncertain, we get a bigger error.

Finally, we consider variations in three stations, 401, 402, and 403, all at Conner's Creek for a 90% input uncertainty in the effluent source strength of fecal coliform. In Figure 8, we display the Relative Standard Deviation vs. downstream distance. The output uncertainty is independent of distance downstream and is considerably less than 90% over the range studied.

Summary

The FAST method has been used to evaluate the relative contributions of uncertainties in the input parameters to the overall uncertainty in water quality predictions on the Detroit River. The FAST method allows an analysis of station by station contributions as well as insight into the river parameters which cause model predictions to deviate from measured data.

References

- Cukier, R.I., C.M. Fortuin, K.E. Shuler, A.G. Petschek and J.H. Schaibly, J. Chem. Phys. 59, 3873 (1973).
- Cukier, R.I., J.H. Schaibly and K.E. Shuler, J. Chem. Phys. 63, 1140 (1975).
- El-Sharkawy, A., R.H. Kummler and C-S Liang, Proceedings of the Engineering Foundation Conference, Niagara-on-the-Lake, Ontario, Canada, November 1983.
- Giffels, Black and Veatch, a CS-806 Report on Quantity and Quality of Combined Sewer Overflows, Vol. II, September 1980.
- Koda, M., G.J. McRae and J.H. Seinfeld, Int. J. Chem. Kin. 11, 427 (1979).

FIGURE 4.

RELATIVE STANDARD DEVIATION % VS. DISTANCE FOR TP AT Y=70.0FT

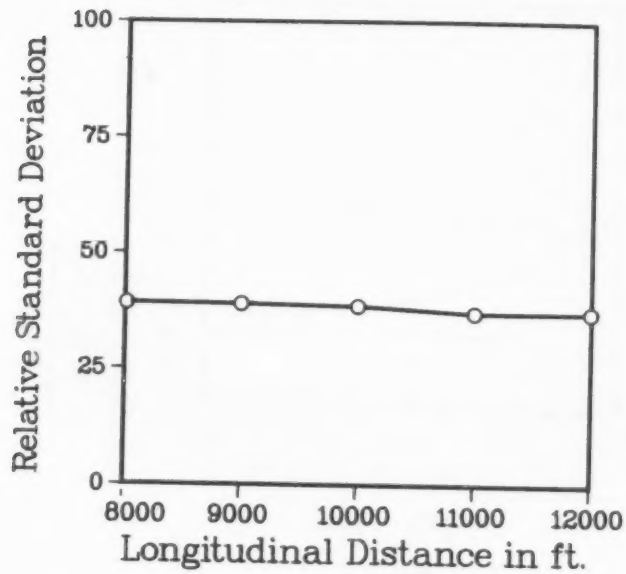


FIGURE 5.

RELATIVE STANDARD DEVIATION % VS. DISTANCE FOR COLIFORM AT Y=70.0FT

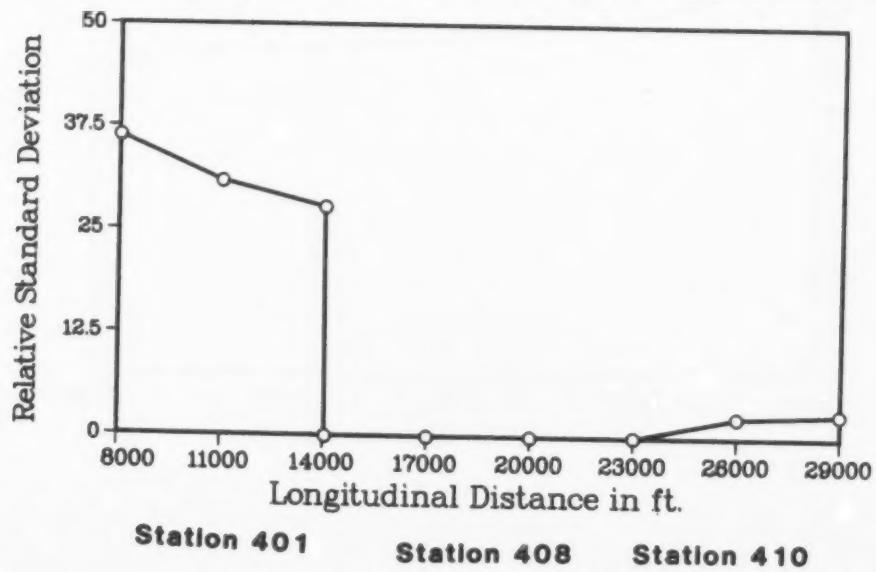
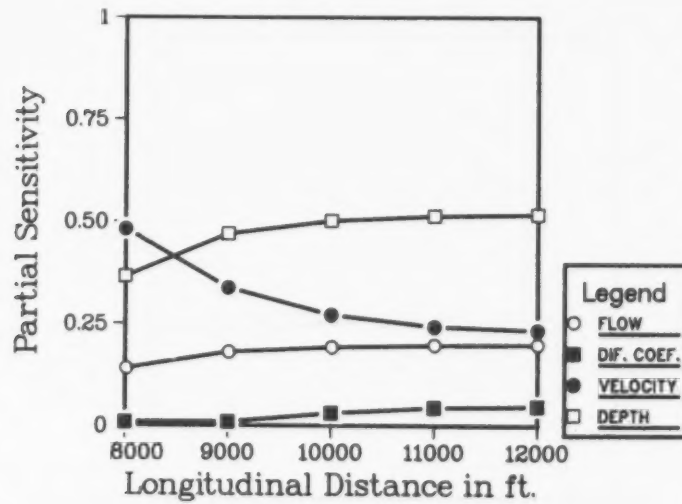


FIGURE 6.

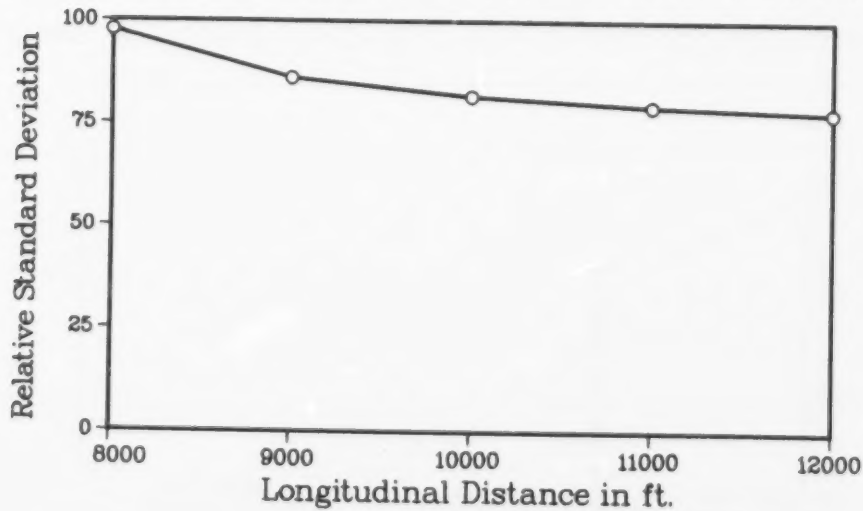
PARTIAL SENSITIVITY VS. LONGITUDINAL DISTANCE FOR TP AT Y=70.0FT.



station 401

FIGURE 7.

RELATIVE STANDARD DEVIATION % VS. DISTANCE FOR TP AT Y=70.0FT



Station 401

Kummler, R.H., J.G. Frith, L-S. Liang and J.A. Anderson, "Uncertainty Analysis in Stormwater and Water Quality Modelling," Proceedings of the SWMM Users Group Meeting, USEPA and McMaster University, Hamilton, Ontario, Canada, September 1981.

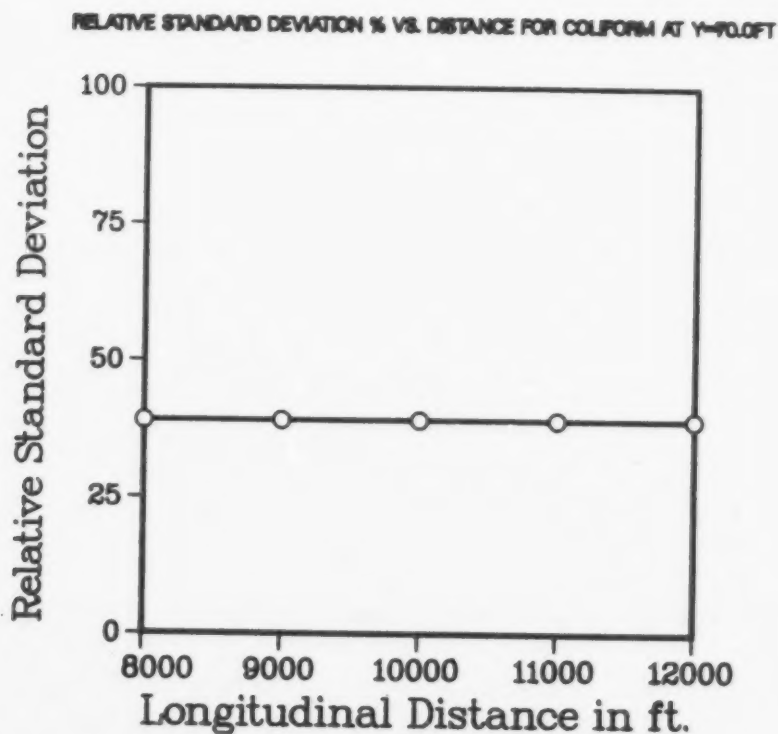
Kummler, R.H., "SWMM Modelling for the Detroit 201 Final Facilities Plan: Final Results," Proceedings of the USEPA SWMM Meeting, Ottawa, Ontario, October 1982.

Liang, Chein-Sung, "Use of Multispectral Remote Sensing Data to Predict the Turbulent Diffusion Coefficient in the Detroit River," Master's Thesis under R.H. Kummler, Department of Chemical Engineering, Wayne State University, Detroit, Michigan (1981).

Liang, Chein-Sung, S. Winkler and R.H. Kummler, "A Gaussian Plume Model of a Two Dimensional River," Symposium on Section 201 Planning; Modeling for Combined Sewer Overflow Abatement, Paper 8d, 91st National AIChE Meeting, Detroit, Michigan, August 1981.

McRae, G.J., J.W. Tilden and J.H. Seinfeld, Computers and Chemical Engineering 6, 15 (1982).

FIGURE 8.



Norbeck, J., Ford Motor Company Technical Report (1983).

Roginski, Gregory T., "A Finite Difference Model of Pollutant Concentrations in the Detroit River from Combined Sewer Overflows," Ph.D. Dissertation under R.H. Kummler, Department of Chemical Engineering, Wayne State University, Detroit, Michigan, 1981.

Seinfeld, J.H., Air Pollution Physical and Chemical Fundamentals, McGraw Hill Book Company, New York (1975).

Weyl, H., Amer. J. Math. 60, 889 (1938).

RAINPAK - A PROGRAM PACKAGE FOR ANALYSIS OF
STORM DYNAMICS IN COMPUTING RAINFALL INPUTS

by

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ABSTRACT

A software package entitled 'RAINPAK' comprising four separate programs has been developed to process rainfall observations in order to estimate storm cell characteristics such as speed and direction of motion, spatial dimensions and growth/decay functions. These estimated properties are then used to model rainfall produced by one or more storm cells. The discretized hyetographs produced by the rainfall model thus incorporate dynamic storm characteristics.

Application of RAINPAK to rainfall events will be covered in another publication.

INTRODUCTION

Drainage network design procedures typically require a dataset describing the 'as-is' hydrological characteristics of the problem catchment. Certain parameters, such as conduit length, conduit slope, conduit roughness and subcatchment area are easily determined; others such as percent imperviousness,

infiltration and subcatchment width, require calibration. Calibration procedures in turn require historical runoff and rainfall time series; the physical drainage network or environmental dataset is used in conjunction with the observed rainfall and runoff time series dataset. By varying the most sensitive environmental parameters, an optimal match between the computed and observed hydrographs is obtained. This procedure is repeated for several observed events until an overall 'best' set of calibration parameters has been determined. Normally, the next step in the design requires the use of a design storm with a prescribed return period as input to the calibrated hydrological model. The output hydrograph is then used to design the drainage system, e.g. sewers, retention ponds, overflow structures, etc. Using this approach, the design hydrographs are assumed to have the same return period originally assigned to the design rainfall hyetographs.

There are several difficulties associated with this procedure, the most important relating to the use of rainfall time series. In most catchment areas there are very few rain-intensity gauges (often there will be none at all). The engineer may adopt a hyetograph from a single gauge, assuming that this temporal rainfall distribution is representative of the entire catchment area. Except for small, relatively impervious catchments, and long, soaking rainfalls, this is not a reasonable assumption. For summer storms and most catchments, the result is erroneous calibration.

Other shortcomings exist in the implementation of the design storm. Design storms are usually derived from long historical records consisting of several types of storms. The synthetic design storm is then applied uniformly over the entire catchment area - an unrepresentative condition in many cases. Finally, the computed runoff hydrograph (usually the peak flow) produced by this design

storm is assumed to have the same return period as the rain hyetograph (usually the total rainfall). This does not take into account the antecedent conditions present in the catchment area, nor the new response time of the problem catchment, altered by virtue of the design or proposed drainage changes. Current methodology obviously makes no provision for storm dynamics or multicellular features apparent in rainstorms. The accuracy of present runoff models has outstripped the accuracy of available input time series. This paper focuses on better ways of determining the input time series.

A program package, RAINPAK, has been written to develop rainfall input and a new design methodology incorporating dynamic storm modelling is introduced. The descriptions of physical processes behind the temporal and spatial distribution of storm rainfall, specifically thunderstorms, derived from raingauge, radar and satellite observations form the basis for most of the concepts used in RAINPAK.

A network of intensity measuring raingauges is used to estimate storm characteristics such as speed, direction and cell growth/decay mechanisms. A new procedure, storm calibration (fitting computed hyetographs to observed hyetographs), is described and applied to observed data. A methodology to develop a numerical storm model from simple hyetographs is presented. This storm model is used to compute rainfall in space over a catchment area. It is suggested that hydrological models can be more accurately calibrated using such improved rainfall representation. The importance of better calibration is obvious. The design of overflow structures, super-sewers, interceptor sewers, etc. is more reliable.

A series of numerical experiments were conducted to determine how well an estimated storm cell shape will reproduce observed hyetographs. Results will be presented elsewhere.

RUNOFF SENSITIVITY TO RAINFALL VARIABILITY

Several hydrologists have studied the effect of storm dynamics on runoff. These studies have included numerical rainfall models, numerical experiments applying synthetic rainfall to a hypothetical catchment, and the application of various rainfall distributions to a dataset describing an actual catchment.

If variable storm patterns are not accurately accounted for, the resulting errors are considered particularly important to computational modelling (Troutman, 1981). Because of these errors, Troutman recommends that model calibration be performed using input data subject to the same kind of errors. By incorporating areally-distributed inputs into a model, taking account of spatial variability. Improved computed hydrographs will result. This is due primarily to the significant effect of distributed rainfall volumes (Beven and Hornberger, 1982).

Idealized numerical rainfall models have been applied to data sets of real and hypothetical catchments, and the subsequent computed runoff analysed (Wilson et al, 1979; Amorocho, 1981; James and Scheckenberger, 1982). Wilson compared computed runoff hydrographs produced by a network of twenty gauges to those produced by one randomly chosen gauge. A moving rainfall model was used to produce 'storms' with an areal mean depth in excess of two inches and moving at twelve mph. The storms ranged from highly random to highly organized spatial distributions and were applied to a 26.5 sq.mi. catchment area in Puerto Rico. The results are tabulated below.

% DIFFERENCE BETWEEN THE USE OF TWENTY GAUGES AND ONE GAUGE

STORM DEPTH		RUNOFF VOLUME		PEAK DISCHARGE	
Mean	Max	Mean	Max	Mean	Max
8	22	13	35	13	52

It may be concluded that when the spatial variability of rainfall is ignored (e.g. by using only one input rainfall time series), computed runoff incurs unacceptably high errors.

Others have examined runoff sensitivity to rainfall variability by applying a temporal rainfall distribution in a static or kinematic fashion (Aron et al, 1982; James and Scheckenberger, 1982). Aron et al examined the sensitivity of computed peak discharge for an actual and hypothetical catchment to temporal and spatial rainfall variations. They concluded that, as drainage areas decrease in size, the sensitivity of peak discharge to time distribution and maximum short duration rainfall intensity increases. It thus becomes increasingly more difficult to estimate peak discharge. A 'late' peaking rainstorm produced computed runoff peaks up to 30% higher than 'early' peaking storms. Travelling storms (i.e. rainfall distributions lagged spatially in time) moving in the general direction and speed of the runoff down the drainage network, produced computed peak discharges 30-35% higher than equivalent stationary storms. These results concur with those obtained by James and Scheckenberger; both sets of authors recommend the use of storm dynamics in the design of drainage systems. Robinson and James (1982) optimized the design of storage units by varying the speed and direction of storms across an urban centre.

In a study conducted in Stevenage, England, Colyer (1981) analysed 21 observed storm events. The peak discharge was computed by the Wallingford Hydrograph Method using hyetographs recorded at two gauging sites individually and in combination. These gauges were 1390 m apart and had a 23 m difference in elevation. Using a dimensionless parameter to represent the degree of fit between observed and computed peak discharge, Colyer found dramatic variations. For example, when comparing computed peak discharge using the 'lower' gauge against the 'upper' gauge the ratio between computed peak and observed peak

varied between 0.354 and 1.479. Thus Colyer concluded that using a single raingauge as input could lead to very unrealistic representations of individual storms over an urban catchment.

It seems quite clear from these numerical and laboratory experiments that the temporal and spatial distribution of rainfall has a marked influence on the computed runoff hydrograph. Moreover, when rainfall is convective, characterized by short duration, high intensities and localised in space, any rainfall-runoff model without an appropriate description of the spatial and temporal character of the rain input may produce unacceptable errors (Wilson et al, 1979).

STORM DYNAMICS

Some storm motion characteristics are obtainable from radar measurements (Huff et al, 1981). Usually storm motion is estimated by tracking the centre of gravity of the rain area (Bellon and Austin, 1977). Difficulties arise because the process is computer-intensive. Radar data are not as available as raingauge data and the available record is relatively short. Thus hydrologists have developed methods to determine cell motion and characteristics from raingauge networks.

Possibly the simplest method of estimating storm movement is to sequentially multiply a rainfall total on a network grid by its distance from the 'x' and 'y' axes. By summing this quantity for all raingauges involved and dividing by the total rainfall, the coordinates of the centre-of-gravity are determined. This method will usually provide a good estimate of general storm motion, but if more than one cell is present, the estimate of speed may be in error. Moreover, this methodology requires that the storm centre passes through the gauging network (Shearman, 1977).

In another technique, an identifiable feature in the hyetograph, such as the start of rainfall, is tracked. Unfortunately, a long time is required to preprocess the hyetographs (Shearman, 1977).

A third technique involves lag correlation analysis (Marshall, 1980). This technique requires a gauging network sufficiently dense that gauges within the same raincell will produce a high correlation coefficient at zero time lag (Niemczynowicz and Jonson, 1981). If the lag time is known or estimated, correlations between pairs of gauges will be improved in the direction of the storm movement. This technique assumes rainfall to be isotropic (i.e. rainfall at any distance from the storm centre is independent of direction). The 'optimum' lag time can be obtained between hyetographs from a correlogram. By repeating this procedure for all pairs of raingauges in the network a set of 'optimum' lag times is obtained. A least squares method is used to determine a single best lag time from the set. A confidence estimate is made by determining total variance due to regression and residual variance.

Shearman (1977) performed several numerical experiments using Marshall's correlation technique. Irregular gauge networks, including outliers, produced significant errors using idealized data. By applying random timing errors of 0, +/-5, +/-10, +/-15 minutes to the lag time at the 15 gauges, the sensitivity to timing errors was determined. For three idealized test storms, the largest error for a 25 sq.km storm cell moving at 0.4 m/s and 225 degrees, was +2.5 m/s and +13/6 degrees.

RAINPAK

RAINPAK comprises four programs: STOVEL, THOR4DPT, THOR4D and THOR3D. STOVEL, THOR4DPT and THOR4D, while having stand-alone capabilities, are usually executed consecutively. THOR3D is a simplified version of the procedures

addressed by the other three programs. RAINPAK uses hyetograph data as input and models storm dynamics. For details on input formats, limitations and sample I/O, refer to the RAINPAK USERS MANUAL (Scheckenberger and James, 1984).

The basic analyses performed by the various programs in RAINPAK can be summarized as follows:

1. Storm Dynamics Analysis (STOVEL)
2. Cell Calibration (THOR4DPT)
3. Hyetograph Synthesis (THOR4D, THOR3D)

STOVEL

STOVEL (STorm VELOCITY) is logically the first program used in a hydrologic analysis. Using at least three hyetographs, STOVEL provides an estimate of the speed and direction of a storm cell. In addition, STOVEL examines patterns of growth and decay associated with the storm lifetimes. As cells move across a study area, optimum relationships are determined which define characteristic cell development.

Thunderstorms display intense, short duration rainfall, rapidly varied spatially. Due to variabilities in wind, terrain (natural and man-made), moisture availability etc., each hyetograph displays variability, but certain features, such as peak intensity and duration, may be identified, even for the more common complex multicell thunderstorm (Huff, 1967).

The drawback to tracking an identifiable feature of a storm is the subjectivity involved, as well as the lengthy processing time (Shearman, 1977).

Thunderstorm cells generally preserve their shape (especially over short distances). Persistence of speed and direction has been observed over several hours (Moses, 1981). STOVEL estimates the speed and direction of a cell by tracking an identifiable feature between gauging stations, noting the relative

timing. In STOVEL the peak intensity was chosen rather than the start or end of rainfall for two reasons: firstly the start and especially the end of rainfall are highly influenced by environmental conditions, much more than the peak intensity; secondly because of the isotropic nature of thunderstorm rainfall, the observed peak intensity is closely correlated with the observed volume of event rainfall at any gauging site.

The Cartesian coordinates of at least three raingauges and the relative time of peak at each gauge are required by STOVEL to estimate the direction and speed of a storm cell. Every combination of three raingauges produces a cell speed vector. When more than three gauges are used, an arithmetic and weighted average of storm cell velocity vectors is determined. The weighted average is based on the inverse of the distance between gauges, i.e., the perimeter of the triad of gauges. The idea is based on the tendency for uniform structure to be strong on short paths but subsequently decrease with increasing path length (Drufuca, 1978). This weighting approach is similar to that used by Kelway and Herbert (1969). When the difference between arithmetic and weighted vectors is large, the inverse perimeter weighted vector is preferred.

In addition to speed and direction, STOVEL determines the locus of the storm cell. The most intense rainfall typically occurs near or at the centre of the cell area. A locus will invariably be produced within the gauging network area using this technique. This will only be a disadvantage when the area defined by the network is considerably smaller than the catchment under analysis. Nguyen et al (1978) proposed a similar approach based on hourly timesteps.

When more than three gauges exist, it is unlikely that the average velocity vector determined by STOVEL will produce peak times identical to those observed. Therefore, the time-to-peak at each rain gauge is recalculated using a least-

squares fit, adjusting computed times closer to those observed. The relative temporal position of the computed peak times is maintained. In order to model a thunderstorm's rainfall intensity variation in time, STOVEL analyzes the peak intensity at each gauge. A non-linear curve is fitted to the peak intensities and their time of occurrence. The curve allows the computed thunderstorm cell to grow and/or decay while moving across a study area.

The output, which can be used directly by one or more of the other programs in RAINPAK without any additional user intervention, is:

- i) date of storm event
- ii) listing of all possible triad combinations, their computed directions, speeds and perimeters
- iii) arithmetic and perimeter-weighted average storm velocity vector and its standard deviation.

The following are output for both the arithmetic and inverse perimeter weighted average storm velocity vectors:

- (iv) hyetograph start time
- (v) time for storm cell to reach first gauge
- (vi) equation of storm track, x, y, intercepts
- (vii) parabolic growth and decay function constants
- (viii) indicator of possible problematical raingauge
- (ix) timing sensitivity analysis

The standard deviation of the average velocity vector, meaningful for two or more triad combinations, is provided in units of degrees and km/h. It is extremely useful for assessing the reliability of the determination.

All gauges in a triad combination producing a storm velocity vector, whose direction is more than one standard deviation from the mean, are flagged as possible sources of timing error. This can also result in several 'accurate'

gauges being flagged. However, gauges with 'significant' timing errors will be flagged in several erroneous vector determinations. A table 'Indicator of Possible Problematical Raingauge' is provided showing all such occurrences. By removing the 'most' erroneous gauge from the analysis, the standard deviation will be reduced, thereby improving the reliability of the determination.

In addition, a timing sensitivity analysis is carried out. The time-to-peak at each raingauge is automatically altered plus and minus one user-specified time increment (this increment is usually the minimum resolution of the hyetograph). This is repeated for every raingauge and the resultant velocity vector and its standard deviation are determined.

THOR4DPT

THOR4DPT is the next program to follow a STOVEL analysis. Using the output from STOVEL, THOR4DPT is used to compute hyetographs at the raingauge locations used in the STOVEL analysis. In the THOR4DPT 'storm calibration' procedure, cellular shape and dimensions are systematically varied to optimize the degree-of-fit between observed and computed hyetographs. The key objective functions used to assess goodness of fit are:

- a) total precipitation
- b) peak rainfall intensity
- c) shape (timing of peak)
- d) duration

The theoretical storm cell shape incorporated in the methodology compares closely with those described by many earlier works. THOR4DPT incorporates the following assumptions:

1. Storm cells are generally oblong, may be approximated by an ellipse with circular being a special case, and remain so throughout their lifetime.

2. Peak rainfall intensity is located at the centre of the specified shape.
3. Rainfall intensity away from the centre varies exponentially, decreasing to a user specified level at the cell boundary (i.e. a circular cell would have an isotropic distribution).
4. Storms can be multicellular, where cells are free to merge and overlap (Kelway and Herber, 1969).
5. Rainfall cells are oriented with their own axes parallel to its motion (NOTE: in a study of over 542 cells in the Montreal area, over 60% conformed to this (Bellon and Austin, 1977)).

The location of the thunderstorm cell is computationally controlled by moving the geometric storm centre by an incremental distance. The computational time increment in THOR4DPT is one minute. The STOVEL determined cell speed and direction are used to define the incremental distance.

During the life cycle of a typical thunderstorm cell, several cell properties constantly change. THOR4DPT has also been provided with four time varying functions:

- 1) peak cell rainfall intensity
- 2) cell areal coverage
- 3) cell speed
- 4) cell direction

The growth/decay of peak rainfall intensity is accounted for by the parabolic relation determined by STOVEL. In THOR4DPT exponential functions can be used to vary both peak rainfall intensity and areal coverage. The other two cell properties addressed are speed and direction; a starting speed and/or direction is specified and then a time rate of change estimated (i.e. acceleration or deceleration and counter-clockwise or clockwise rotation).

Two types of data are required, meteorologic and geographic. Meteorologic input consists of:

1. peak cell intensity
2. cell shape and dimensions

3. boundary rainfall intensity
4. area, speed and direction as time dependant functions
5. cell speed
6. cell direction
7. Cartesian coordinates at start time of cell centre
8. peak growth/decay modulating function.

The first three parameters are revised by the storm calibration procedure. Parameter 4 is optional. STOVEL provides parameters 5 through 8.

Starting values for the first three parameters may be estimated from hyetographs prior to calibration.

For a given cell, the hyetograph observed at any number of raingauges will have approximately the same duration (exceptions include gauges located on the periphery of a storm cell). Also the peak intensities of the raingauges located nearest to the determined cell track can be used for an initial estimate of the peak rainfall intensity of the cell.

The two remaining input parameters, the transverse cell dimension (only applicable to ellipses) and the cell boundary rainfall intensities, can be considered to be calibration parameters. Initial values of these parameters can be estimated by using: transverse dimension = longitudinal dimension and cell boundary rainfall intensity equal to one-twentieth of peak intensity. Although arbitrary, these parameters converge reasonably quickly to 'acceptable' values. A comprehensive chart has been prepared which describes the use and effect of these calibration parameters (the chart appears in the RAINPAK - Users Manual).

The only geographic data required by THOR4DPT are the raingauge Cartesian coordinates, identical to that required by STOVEL.

The output from THOR4DPT consists of computed hyetographs and rainfall totals at the specified raingauge locations. If the computed and observed

hyetographs do not compare favourably, appropriate changes should be made to the calibration parameters. Of course, discrepancies will always exist between observed and simulated hyetographs, especially in densely gauged areas. Some of the possible reasons are:

1. the idealized cell rainfall distribution
2. turbulence caused by wind eddies around geographic features, both natural and man-made
3. incomplete, irregular merging of cells
4. data timing and recording errors
5. cells missing gauges entirely

THOR4D

THOR4D is identical to THOR4DPT but rather than computing point rainfall intensities, THOR4D produces time and areally-averaged hyetographs for sub-areas. Thus THOR4D determines the average rainfall hyetograph falling over every subcatchment within a larger watershed for use as input for a hydrological program such as the SWMM. The higher the level of discretization of the watershed, the higher the level of discretization of the storm. By utilizing the meteorologic data determined by STOVEL and THOR4DPT, in addition to geographic input, THOR4D develops sub-basin wide averaged rainfall input incorporating all of the characteristics of kinematic storms.

Subcatchments are laid out on the same Cartesian plane as the raingauges in the STOVEL analysis. Subcatchments can take on an infinite number of irregular shapes, however these shapes can become increasingly regular, especially in urban centres, where drainage pipe networks rather than physical terrain usually dictate subcatchment delineation. Often, for the usual North-American grid-style urban centres, subcatchments are approximately rectangular.

Rectangular subcatchments have been adopted as the most practical areal representation in THOR4D thereby reducing the data abstraction effort substantially.

In THOR4D, point rainfall intensity is averaged areally by double numerical integration. Trapezoidal integration is used when integrating the theoretical exponential 'raincone' because the more complex numerical integration routines investigated showed no significant improvement in accuracy despite their higher computing costs.

Numerical experiments were carried out on the number of points representing a rectangular subcatchment. This is the most computationally intensive subroutine in THOR4D. Sensitivity analysis showed a small increase in accuracy when the number of divisions along a subcatchment dimension increases beyond 5.

The meteorological data required by THOR4D is identical to that required by THOR4DPT and provided by STOVEL. The geographic data describing the subcatchments must be acquired from maps. Five data items are required to represent subcatchment orientation and configuration in a Cartesian plane.

The first of the items are the co-ordinates of the subcatchment centroid. The centroid of a rectangle is simple to obtain, however highly irregular shapes present difficulty. Subjective estimates should generally be adequate (Scheckenberger and James, 1984). Nonetheless, a sensitivity analysis on the location of the areal centroids to determine their influence on runoff characteristics is recommended.

The other four data items required are weighted widths in the N-S, E-W and NW-SE, NE-SW directions. Since very few subcatchments will be aligned in a purely N-S or NW-SE orientation, it proved convenient to adopt two co-ordinate systems in THOR4D. The assumption is that when several subcatchments exist in a

watershed, their orientation will best be approximated by a mixture of both co-ordinate systems. Practice has shown that usually more than two-thirds of the subcatchments are common to both co-ordinate systems (Scheckenberger and James, 1984).

THOR4D output comprises SWMM formatted hyetographs at a user specified timestep, one hyetograph for every subcatchment. It was necessary to change the SWMM code to accept more than six hyetographs.

THOR3D

Unlike the other programs in RAINPAK, THOR3D does not necessarily require observed rainfall for input. THOR3D develops design storms. Typical use of THOR3D would be to develop kinematic rainfall input for a rainfall/runoff model. Critical storm speeds, directions and distributions of rainfall can be determined which would significantly aid in the design of safer, more efficient drainage networks. Another possible use of a slightly revamped THOR3D is for flood warning; real-time analysis of rainfall from remote raingauges could produce rainfall input for a flood model.

In THOR3D the dimension transverse to storm motion is infinite. This shape is more conservative from a flood design standpoint.

Unlike THOR4D, the peak intensity may be located anywhere within the cell, e.g. as in the quartile rainfall distributions (Huff, 1967). Huff showed that the location of peak rainfall intensity within a rain period can be a regional characteristic. The location of the peak can be critical for drainage network design. Since the dynamics of flow in drainage networks, particularly large urban centres, is unique to the system, various networks can be more or less affected by the same temporal and spatial distribution. THOR3D affords the

engineer the flexibility to vary the location of the peak intensity to determine the most severe design case unique to the particular drainage network under analysis. The position of the peak intensity is specified by the ratio of the time of hyetograph recession to the time of hyetograph rise.

THOR3D is not used to simulate observed events: no provision has been made to synchronize the timing of computed and observed rainfall movement over a catchment area. In THOR3D the leading edge of the line-storm is assumed to be located at the outermost cardinal boundary of the catchment area at computation time zero. THOR3D internally determines eight outermost catchment boundaries in the N, NE, E, SE, S, SW, W and NW directions respectively.

Similar to THOR4D, THOR3D requires a minimum 'cutoff' rainfall intensity at the leading and trailing edges of the line storm. This parameter is used in conjunction with the time of hyetograph rise to determine the exponential rising constant.

In THOR3D the minimum rainfall intensity is used in a pseudo-calibration manner. For a given total precipitation during a rainfall event, a range of peak intensities within the same storm duration can be obtained by varying the minimum rainfall intensity (i.e. the higher the minimum rainfall intensity, the lower the peak rainfall intensity and vice-versa). This is accomplished internally in THOR3D, through the use of continuity equations which preserve total precipitation as well as the ratio of time of recession to time of rise. In THOR3D, the user specifies:

- 1) Total precipitation (TOPR)
- 2) Time of recession/Time of rise (RCSNFR)
- 3) Minimum rainfall intensity as a fraction of the peak intensity (PKFRI)

and either

4) Peak one-minute rainfall intensity (PEAK)

or

4) Time of hyetograph rise (TMRISE)

Provision has been made in THOR3D to simulate more than one line storm during a single event, analogous to the multi-cellular capability of THOR4D. In THOR3D each line storm must be completely parameterized and a lagtime is required to separate cell occurrences. Again, as in THOR4D, line storms are allowed to merge and overlap with one another. This is a feature in THOR3D which can be used to pre-wet a watershed to various degrees of saturation prior to the application of a design storm, similar to continuous hydrologic modelling. With slight modifications, THOR3D, could produce a continuous dynamic rainfall record for every subcatchment, thereby replacing synthetic design storm methodology.

The spatial orientation and configuration of subcatchments are represented numerically in exactly the same manner as in THOR4D. The exponential growth-decay of the peak intensity of the line storm is also provided in THOR3D.

The output format and usage of THOR3D is identical to that of THOR4D. The engineer can examine the effects of storm dynamics on urban drainage system response. Runoff characteristics such as peak flow, volume of runoff, hydrograph shape and system storage are highly influenced by storm characteristics such as speed, direction and time distribution. Design engineers can use THOR3D to perform a comprehensive sensitivity analysis determining which storm characteristics are most critical to the runoff characteristics for the drainage network under analysis.

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BIBLIOGRAPHY

1. Amorochio, J., "Stochastic Modelling of Precipitation in Space and Time", presented at the International Symposium on Rainfall-Runoff Modelling, Mississippi State University, 20 pp. May 1981.
2. Aron, G., Riley, K.A. and Kibler, D.F., "Sensitivity of Urban Flood Peaks to Rainfall Intensity Patterns", 15 pp., 1982.
3. Bellon, A., and Austin, G., "The Real-Time Test and Evaluation of 'SHARP'. A Short Term Precipitation Forecasting Procedure", Quebec, 19 pp. March 1977.
4. Beven, K.J. and Hornberger, G.M., "Assessing the Effect of Spatial Pattern of Precipitation in Modelling Stream Flow Hydrographs", Water Resources Bulletin, Vol. 18, No. 5, October, 1982.
5. Colyer, P.J., "The Variation of Rainfall Over an Urban Catchment", presented at the Second International Conference on Urban Storm Drainage, Illinois, June, 1981, Wallingford, England, Report No. IT211, pp. 19-26, January 1981.
6. Drufuca, G., and Rogers, R.R., "Statistics of Rainfall over Paths From 1 to 50 km", Atmospheric Environment, Great Britain: Vol. 12, pp. 2333-2342, 1978.
7. Huff, F.A., "Time Distribution of Rainfall in Heavy Storms", Water Resources Research, Vol. 3, No. 4, pp. 1007-1019, 1967.
8. Huff, F.A., Vogel, J.L. and Changnon, Jr. S.A., "Real-Time Rainfall Monitoring - Prediction System and Urban Hydrologic Operations", Journal of the Water Resources Planning and Management, ASCE, Vol. 107, No. WR2, pp. 419-435, October 1981.
9. James, W., and Scheckenberger, R.B., "Application of a Kinematic Storm Model to Runoff Modelling", EOS, Vol. 63, No. 13, pp. 323, May 1982.
10. Kelway, P.S. and Herbert, S.I., "Short-Term Rainfall Analysis", Weather, Vol. 24, pp. 342-353, 1969.

11. Marshall, J.K., "Local Considerations Affecting the Choice of Design Storm Frequency", Storm Input, p. 19-30, 1980. (uncredited manuscript).
12. Moses, F., "Convective Cloud-Tracking Techniques for Estimating Precipitation from Geostationary Imagery", Preprint Fourth Conference on Hydrometeorology, Reno, Nevada, pp. 151-158, October 7-9, 1981.
13. Niemczynowicz, J. and Jonsson, O., "Extreme Rainfall Events in Lund 1979-1980", Nordic Hydrology, Vol. 12, pp. 127-142, 1981.
14. Nguyen, V., McPherson, M.B., and Rousselle, J., "Feasability of Storm Tracking for Automatic Control of Combined Sewer Systems", ASCE, New York, Technical Memorandum No. 35, 29 pp. November 1978.
15. Robinson, M.A. and James, W., "Continuous SWMM Modelling of Hamilton Summer Stormwater Including Certain Quality Indicators - Preliminary Output Time Series Using Discrete-Event Calibration for Non-Industrial Areas", McMaster University, Hamilton, Ontario, Canada, January 1982.
16. Scheckenberger, R.B. and James, W., "RAINPAK - User's Manual", to be published by CHI Publications, 1984.
17. Shearman, R.J., "The Speed and Direction of Movement of Storm Rainfall Patterns", Meteorological Office, Bracknell, Berkshire, England, March 1977.
18. Troutman, B.M., "The Effect of Input Errors in Using Precipitation Runoff Models for Runoff Prediction", International Symposium on Rainfall-Runoff Modelling, sponsored by the WRR, 12 pp. May 1981.
19. Wilson, C.B., Valdes, J.B. and Rodrigues, Iturbe, I., "On the Influence of the Spatial Distribution of Rainfall in Storm Runoff", Water Resources Research, Vol. 15, No. 2 pp. 321-328, April 1979.

DEVELOPMENT OF FLOWS AND LOADS FOR
STEADY-STATE ESTUARY MODELS:
TAMPA BAY CASE STUDY

By
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INTRODUCTION

An intensive wasteload allocation study of the Tampa Bay region is currently being undertaken by the Florida Department of Environmental Regulation. Previous effluent standards were typically set at "zero discharge" for both industry and sanitary sewage treatment plants. This wasteload allocation study being undertaken by the DER is the first comprehensive analysis of the eutrophication problem of Tampa Bay and is the first of its kind within the State of Florida.

The study is using the University of South Florida's (USF) Tampa Bay Model (1). It is an intratidal estuary model which simulates hydrodynamics and mass transport over relatively short time intervals within the 12.4-hour tidal cycle. Because of the prohibitive computer costs associated with running such a model with long-term, time-variable nonpoint source inputs, the model is being used instead for steady-state analyses. It is the purpose of this paper to describe the methodology used to calculate the flows and loads required as input to the steady-state model (2).

This methodology can be applied to other estuary drainage areas which have adequate databases to determine flows and loading factors. It is important to note that the types of data used in this study are available to the general public for other areas all over the United States.

STUDY AREA DESCRIPTION

The Tampa Bay study area presented in Figure 1 consists of three distinct bays: Old Tampa Bay, Hillsborough Bay and the main portion of Tampa Bay. More than 50 percent of the 1,785 square mile (sq mi) drainage area drains into Hillsborough Bay. Most of the flow from this area is regulated by the Hillsborough Reservoir, a water supply impoundment which is located at the

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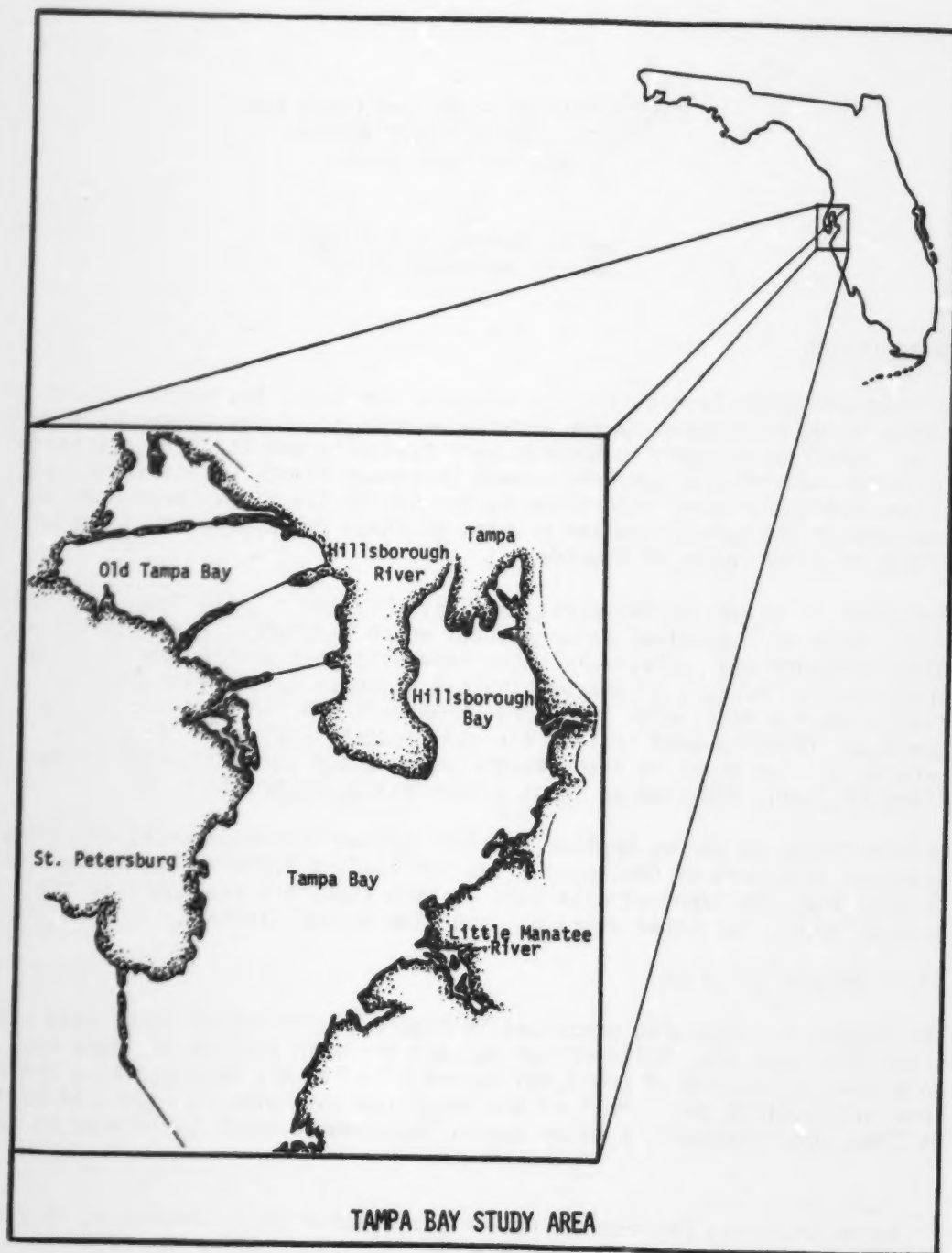


FIGURE 1

mouth of Hillsborough River. Tampa Bay also has an unusual characteristic. Due to the natural phosphate deposits in the area which is inherent in the runoff, algal growth throughout the Bay is limited not by phosphorus but instead by nitrogen.

The flows and loads for input to the estuary model were calculated for the 38 points shown in Figure 2. These points represent not only river confluences but also points of stormwater inflow. The flows and loads were then used as input to the grid system of the USF Model shown in Figure 3.

DESCRIPTION OF METHODOLOGY

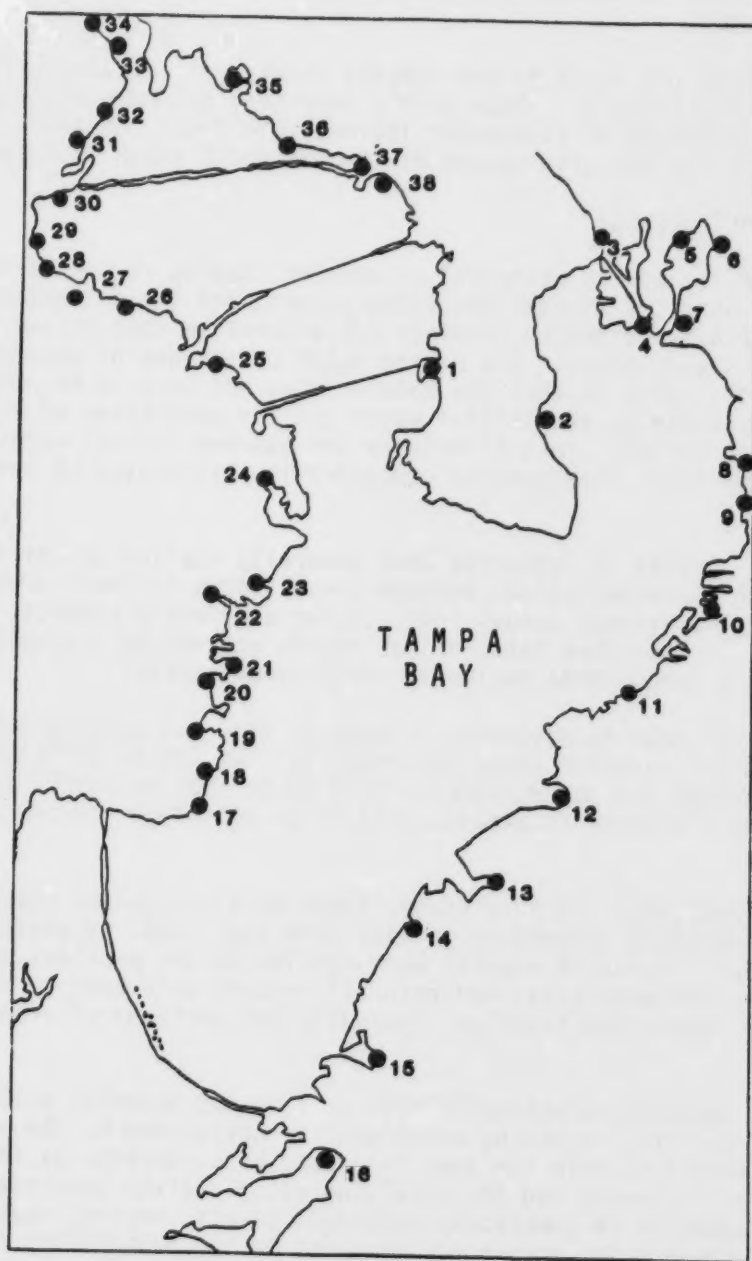
Introduction. Point source wasteload allocation studies for estuaries are typically based upon low streamflows (7Q10 values) and the assumption of continuous, constant wastewater loadings for a duration that allows the system to reach steady-state. One of the major advantages of steady-state estuary modeling studies is that the modeler does not have to be concerned about accurately defining the initial water quality conditions at the start of the simulation period. This is because the assumed initial conditions do not affect the final steady-state concentration calculated by the estuary model.

Nonpoint source studies of estuaries have generally applied one of two types of loading characteristics: average annual loads or loads produced by design storms. The average annual loads cannot accurately predict "worst case" impacts. On the other hand, design storms may not be appropriate for areas requiring a longer time period to reach steady-state.

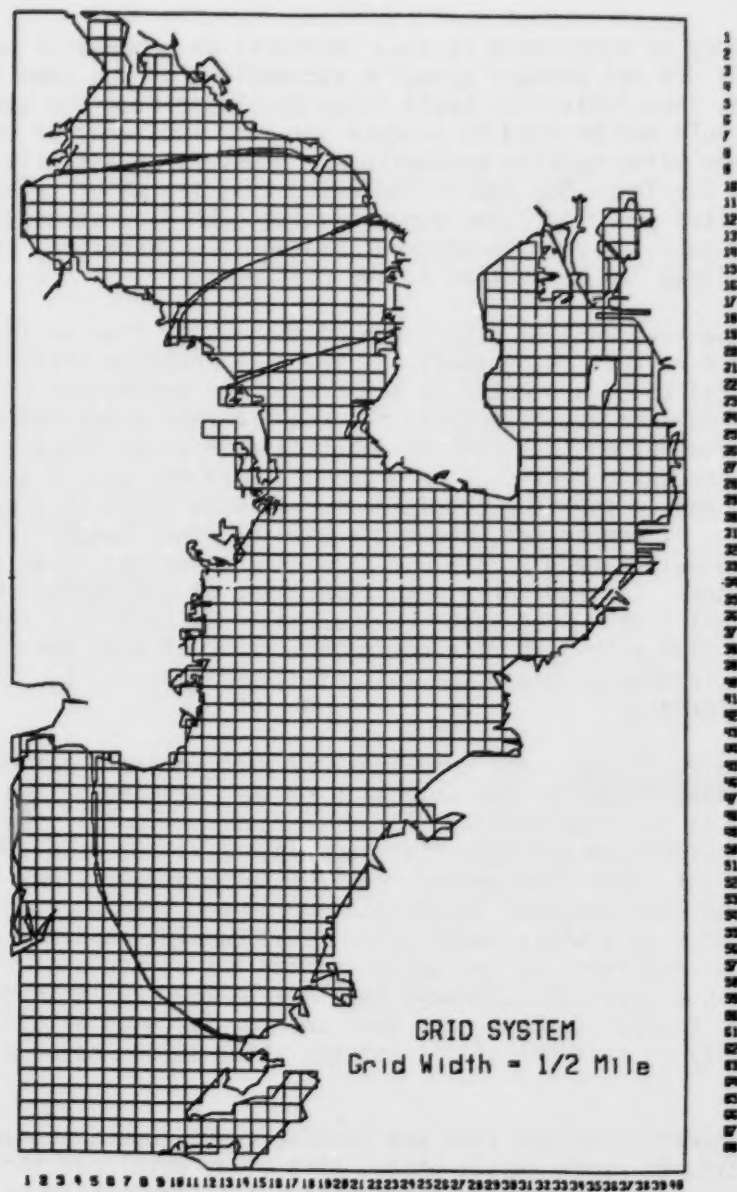
In this study, we chose to determine a range of flow and load values that would encompass all possible study alternatives. We set up different recurrence intervals and the different duration periods to reach steady-state conditions. Several assumptions had to be addressed concerning these parameters.

Flow-Frequency Analysis. In this study, flows were calculated based on the pervious and impervious percentage in each land use. USGS streamflow gaging statistics (inches of runoff) were applied to the pervious sections of the Tampa Bay drainage area, and rainfall records were used to develop runoff from the impervious fraction, typically for portions of urban land uses only.

Because of the positive relationship between flow and nonpoint pollution load (3), the resulting receiving water quality calculated by the estuary model can be assumed to have the same frequency of occurrence as the flow condition. The flow event and the resulting water quality condition in the estuary are assumed to be completely dependent events, meaning that the probability of their joint occurrence is equal to the probability of either event occurring. These assumptions are critical to the assessment of the recurrence interval associated with estuary model projections of water quality impacts from nonpoint pollution loadings.



NODES REPRESENTING INFLOW POINTS FOR BASIN FLOWS AND LOADINGS
FIGURE 2



TAMPA BAY HALF MILE GRID SYSTEM

FIGURE 3

The frequency of occurrence is also important on a bay-wide scale. If the entire area did not produce synoptic streamflows of the same frequency of occurrence, then individual basin flows developed from the statistical analysis could not be used to produce the same frequency of occurrence for the bay-wide water quality projections. However, an analysis of streamflow records in the Tampa Bay region indicated that the major tributary basins have exhibited synoptic flows approximating each flow statistic (i.e., 2-year, 10-year, 25-year recurrence intervals). Therefore, statistically generated flows can be applied to the USF Model.

Ideally, the frequency of occurrence of the design flow period for estuaries should reflect the spatial and temporal characteristics of the beneficial use(s) to be protected or achieved. The protection of certain beneficial uses (e.g., fisheries) may require that a minimum dissolved oxygen concentration of 5 mg/l be maintained in every segment of the estuary for the entire year. This would indicate the use of a relatively infrequent design low flow condition to minimize risks of significant use impairment. By comparison, the protection of other beneficial uses may not require the maintenance of the 5 mg/l dissolved oxygen level at all times and locations. Instead, only a survival-type of environment need be maintained at all times. The latter situation lends itself to the use of a design low flow with a shorter recurrence interval than the first. The recurrence intervals chosen for this study were the 2-, 5-, 10-, 20- and 25-year intervals.

Design Flow Durations. For the Tampa Bay wasteload allocation study, the most important factor in the selection of the duration of the design flow conditions is the time required for different sections of the estuary to reach an equilibrium concentration under steady-state conditions. USF has found that the simulation period required for concentrations in the estuary to reach equilibrium under steady-state flow and loading conditions is on the order of 3 to 4 weeks based on previous baywide studies. Time to equilibrium conditions is typically shorter for portions of the upper estuary (e.g., upper Hillsborough Bay) due to the smaller storage volumes and shorter transport times, and even shorter for studies of nearfield water quality in sections of the estuary which adjoin major discharge points.

Since the duration of the flow and loading input should correspond to the required time to reach steady-state, this study developed streamflow estimates for three different durations which will permit both nearfield and farfield assessments: 7-day, 15-day, and 30-day. The 7-day flow condition permits nearfield assessments, while the 30-day flow condition is used for baywide assessments. The 15-day value is appropriate for intermediate scale assessments in cases where a 30-day response time is excessive.

Thunderstorm Scenario. Nearfield areas which adjoin the urban areas draining into Tampa Bay are likely to experience the greatest water quality impact from urban runoff slug loads. Due to the stabilizing influence of impervious cover, urban runoff volumes tend to be relatively similar throughout the wet season whereas nonurban runoff volumes tend to be higher

in August and September due to the increase in soil moisture and saturated pervious areas that occur as the wet season progresses.

Therefore, the most critical portion of the wet season for urban runoff impacts is the month of June when nonurban flows are relatively low and rainfall is relatively high. In order to assess the potential impacts from urban runoff early in the wet season, an isolated thunderstorm flow scenario (3-hr/6-month design storm) was developed for testing with the estuary model.

The thunderstorm scenario assumes that urban runoff will be produced by a short duration/high intensity rainstorm in subbasins adjoining the estuary. The actual USF Model simulation of the thunderstorm impacts is dynamic in nature. The storm loads are applied to boundary conditions representing average June flow conditions. Once the storm event has ended, the average June flows will again be simulated, using as initial conditions the slug concentrations derived from the storm. This will allow the simulation to track the impact of the thunderstorm loads after the conclusion of the storm event.

DEVELOPMENT OF FLOWS AND LOADS FOR ESTUARY MODEL INPUT

Land Use Characteristics. The calculations of the flows and loads were based upon individual land uses within each river basin. The land use delineation was chosen for two reasons. First, the water quality data available for the rivers in this area are highly biased by land use characteristics of the basin (e.g., phosphate mines in the Alafia River basin). The application of these data to other ungaged basins would require similar land uses, soil and streamflow characteristics within the two basins. Second, the calculation of the flows and loads by land uses permits a more accurate projection of future land use impacts and the reductions in loadings due to the implementation of best management practices. The existing land uses within the Tampa Bay drainage area are presented in Table 1. More than 75 percent of the basin is covered by undeveloped land.

Flow Calculations. The flow calculations were based on the pervious and impervious areas of each land use. Rainfall statistics were used to calculate the runoff from the impervious areas (typically in urban areas only) and streamflow gaging statistics were used to calculate the runoff from the pervious areas. Rainfall records were analyzed to coincide with the duration/frequency scenarios set for the streamflow statistics.

The streamflow statistics were based upon the data available for the eight USGS stations presented in Table 2 and Figure 4. Drainage areas ranged from 35 to 650 square miles. Land uses in each basin were typically rural/agricultural. The soil characteristics presented in Figure 4 were one of the criteria used to assign the gaged basins to ungaged basins.

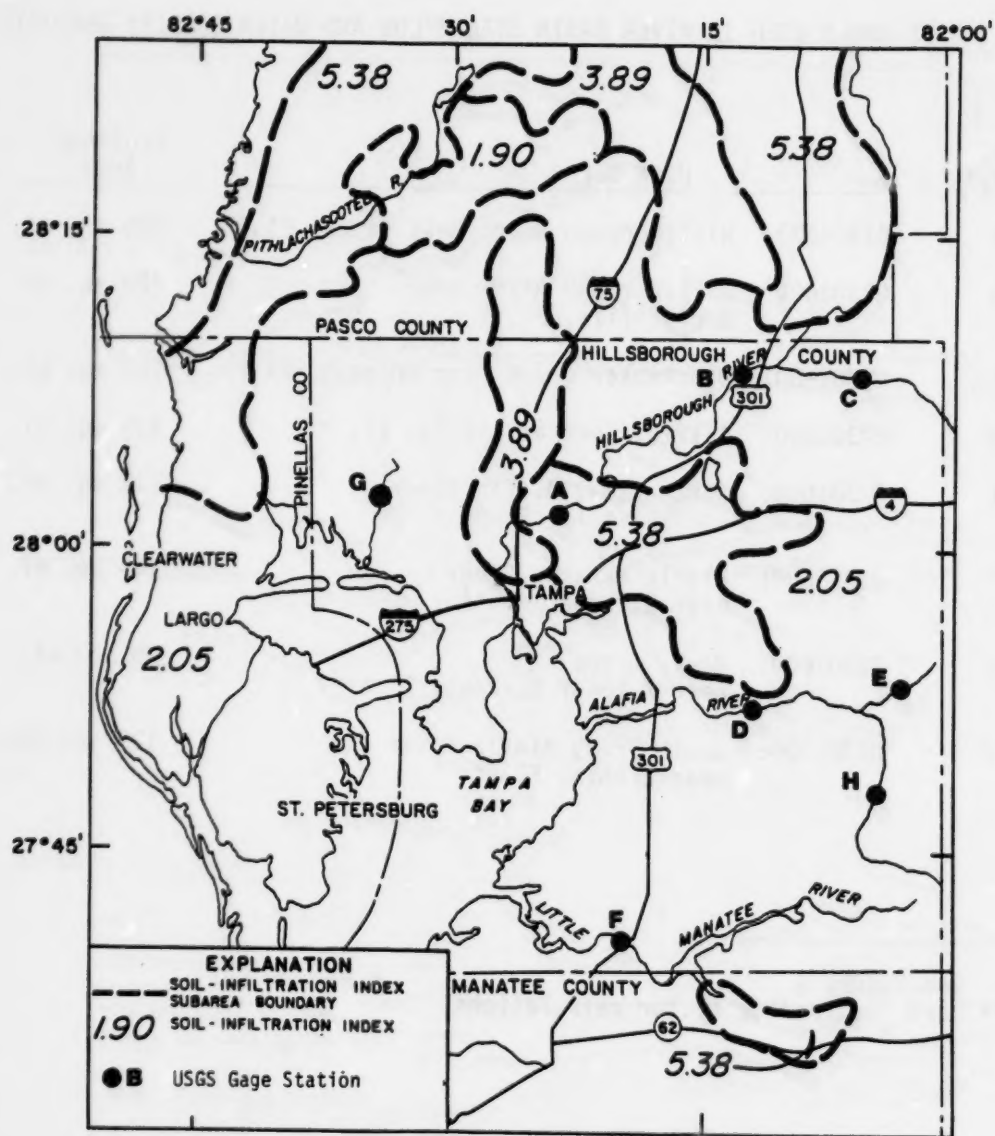
The statistical analysis package was accessed through the STORET database at the EPA Computer Facility in Research Triangle Park, NC. The high and

TABLE 1

EXISTING LAND USE WITHIN
TAMPA BAY DRAINAGE AEA (1,785 sq. mi.)

<u>Land Use</u>	<u>% of Total Area</u>	
Forest	16.6	
Cropland	17.6	
Pasture	38.9	76.8%
Other Rural	3.7	
Urban Residential (Single & Multi-Family)	14.2	
Urban Commercial	5.5	23.2%
Urban Industrial	1.5	
Urban Other (Recreational and Open Space)	2.0	
TOTAL:	100.0%	

Note: This table does not include water surface area for Tampa Bay.



TAMPA BAY REGION MAP SHOWING USGS STREAMGAGES AND
SOIL INFILTRATION INDEX

FIGURE 4

SOURCE: LOPEZ AND WOODHAM, 1983

TABLE 2

USGS GAGES USED IN RIVER BASIN STREAMFLOW AND WATER QUALITY ANALYSIS

Map Key*	USGS Gage	Drainage Area
A	02304500: Hillsborough River near Tampa, Fl.	650 sq. mi.
B	02303000: Hillsborough River near Zephyrhills, Fl.**	220 sq. mi.
C	02302500: Blackwater Creek near Knights, Fl.	110 sq. mi.
D	02301500: Alafia River at Lithia, Fl. **	335 sq. mi.
E	02301000: North Prong Alafia River at Keyville, Fl.**	135 sq. mi.
F	02300500: Little Manatee River near Wimauma, Fl.**	149 sq. mi.
G	02307000: Rocky Creek near Sulphur Springs, Fl.**	35 sq. mi.
H	02301300: South Prong Alafia River near Lithia, Fl.**	107 sq. mi.

* See Figure 4

** Used for loading factor calculations

low flow statistics were developed based upon a Log-Pearson Type III distribution. It should be noted that the STORET flow data and statistical package are available to the general public and can be accessed with relative ease.

The flows generated by the statistical package were converted to inches of runoff. This allowed us to use the individual land use acreage, the duration of the flow and loading averaging period, and the inches of runoff to calculate the flow for each particular input point to the model. The total flow for any of the six land uses was the weighted average of the pervious and impervious flows of the land use. The total flow for the input points was then the summation of the flows from the individual land uses.

Load Calculations. Loading factors were developed for each of the land uses for each of the following constituents: total phosphorus, ortho-phosphorus, organic phosphorus, total nitrogen, ammonia, TKN, organic nitrogen, nitrate-nitrite, BOD₅, and dissolved oxygen. The USEPA Nationwide Urban Runoff Program (NURP) database for the Tampa Bay area was used to develop the loading factors for the urban land uses (4, 5). The USGS NASQAN database was used to develop the loading factors for the rural/agricultural runoff. Once the factors were developed, they were verified by applying them to the USGS Urban Watershed study area (6, 7, 8) and comparing our final calculations with the observed data.

The NURP data were acquired from 5 single land use watersheds varying in size from 9 to 194 acres. The USGS NASQAN data were from the six stations specified in Table 2. The measured concentration datasets were checked for a normal or log-normal distribution based upon the one-sided Kolmogorov-Smirnov (K-S) statistical test. Once the distribution was established, mean concentrations for each constituent were calculated based upon the distribution.

The loading factors developed for the rural/agricultural land use at each of the six USGS stations are presented in Tables 3 and 4 for the wet season and dry season, respectively. The factors for the wet season were applied to the high flow conditions and the dry season factors were applied to the low flow conditions. The factors were then assigned to the ungaged basins dependent upon similar hydrologic characteristics and other basin characteristics (7, 8, 9). These assignments are present in Table 5.

The final urban land use loading factors are presented in Table 6. The test basins listed in the table are the NURP watersheds which we used to develop these factors. These factors were applied to the runoff from both the pervious and impervious areas of each land use.

The validity of the urban land use loading factors was verified with the data from a USGS Urban Watershed study in the Tampa Bay region. The 9 USGS test watersheds consisted of mixed land uses and ranged in size from 0.45 to 3.45 sq mi. Three statistical tests were used to determine the goodness-of-fit of the calculated loadings values with the observed values:

TABLE 3

CONSTITUENT	MEAN CONCENTRATION FOR USGS RIVER BASIN DATABASE: WET SEASON					
	LITTLE MANATEE R.	N. PRONG ALAFIA R.	S. PRONG ALAFIA R.	ALAFIA R. AT LITHIA	HILLS. R. AT ZEPHYRHILLS	ROCKY CREEK
Ortho-P (mg/L as P)	0.50/LN* (27)	6.66/N (10)	2.08/N (9)	4.56/N (16)	0.80/LN (29)	0.18/LN (23)
Total P (mg/L as N)	0.53/LN (27)	6.84/N (10)	2.19/N (9)	4.74/N (16)	0.84/LN (34)	0.21/LN (23)
Ammonia N (mg/L as N)	0.06/N (21)	0.59/LN (10)	0.06/LN (9)	0.12/LN (15)	0.29/LN (36)	0.18/LN (21)
TKN (mg/L as N)	0.85/N (21)	1.26/LN (10)	0.82/LN* (9)	0.98/LN (14)	1.16/LN (34)	1.02/LN (18)
Nitrite + Nitrate-N (mg/L as N)	0.18/N (32)	2.76/LN (10)	0.18/LN (9)	1.41/LN (15)	1.18/LN (39)	0.18/N (27)
Total N (mg/L as N)	1.05/N (21)	3.90/LN (10)	1.00/LN* (9)	2.25/LN (14)	2.27/LN (34)	1.21/LN (18)
BOD ₅ (mg/L)	1.78/LN (4)	-	-	-	1.52/LN* (3)	1.40/N* (3)
Dissolved Oxygen (mg/L)	6.3 /N (23)	5.4 /N (9)	6.9 /N (9)	6.6 /N (15)	5.7 /N (32)	3.9 /LN (23)
Flow (cfs)	147.2 /LN (35)	221.9 /LN (11)	143.3 /LN (9)	446.0 /LN* (18)	241.2 /LN (41)	50.7 /LN (34)

NOTES:

1. KEY: Mean Concentration/Type of Distribution
(Number of Observations) where N = Normal Distribution and LN = Lognormal Distribution
2. Means are based upon the period 1976 - 1981 for Alafia River gages and the period WY 1969 - WY 1981 for all other gages.
3. Distribution which did not pass the one-sided K-S test for 95% confidence interval is indicated by * (e.g., N*)

TABLE 4

MEAN CONCENTRATION FOR USGS RIVER BASIN DATABASE: DRY SEASON

CONSTITUENT	LITTLE HATTEE R.	N. PRONG ALAFIA R.	S. PRONG ALAFIA R.	ALAFIA R. AT LITHIA	HILLS. R. AT ZEPHYRHILLS	ROCKY CREEK
Ortho-P (mg/L as P)	0.32/N (62)	6.83/N* (24)	1.92/N (19)	4.34/N (37)	0.60/LN (59)	0.09/LN (55)
Total P (mg/L as P)	0.36/LN* (62)	7.35/N* (24)	2.01/N (19)	4.49/N (37)	0.60/LN (67)	0.12/LN (55)
Ammonia-N (mg/L as N)	0.05/LN (46)	1.40/N* (24)	0.07/LN (18)	0.28/LN* (37)	0.59/LN* (78)	0.20/LN (46)
TKN (mg/L as N)	0.59/LN (42)	1.85/LN (24)	0.65/LN (18)	0.96/LN (36)	1.21/LN* (75)	0.91/LN* (38)
Nitrite + Nitrate-N (mg/L as N)	0.26/LN* (74)	3.45/LN (24)	0.24/N (19)	2.10/LN (37)	1.47/LN* (82)	0.22/N* (63)
Total N (mg/L as N)	0.84/LN (42)	5.35/LN (24)	0.90/LN (18)	3.05/LN (36)	2.71/LN* (75)	1.11/LN (36)
BOD ₅ (mg/L)	0.77/N (7)	0.9 /N* (2)	-	1.50/N* (3)	0.50/LN (8)	1.55/N (11)
Dissolved Oxygen (mg/L)	7.4 /N (50)	6.7 /N (22)	8.4 /N (18)	8.2 /N (32)	7.0 /N (68)	4.0 /LN (43)
Flow (cfs)	78.1 /LN (83)	93.1 /LN (26)	69.9 /LN (20)	240.0 /LN (38)	119.0 /LN (38)	20.2/LN (76)

NOTES:

- KEY: ----- Mean Concentration/Type of Distribution
(Number of Observations) where N = Normal Distribution and LN = Lognormal Distribution
- Means are based upon the period 1976 - 1981 for Alafia River gages and the period WY 1969 - WY 1981 for all other gages.
- Distribution which did not pass the one-sided K-S test for 95% confidence interval is indicated by * (e.g., N*)

TABLE 5

ASSIGNMENT OF USGS WATER QUALITY GAGES TO
RURAL-AGRICULTURAL SECTIONS OF TAMPA BAY RIVER BASINS

<u>Inflow Point*</u>	<u>River Basin</u>	<u>USGS Gage</u>
1 & 2	Interbay E & W	Rocky Creek
3	Hillsborough River Upstream of Reservoir	Zephyrhills
3	Hillsborough River Downstream of Reservoir	Rocky Creek
4 & 5	Seddon Channel & Ybor City	Rocky Creek
6	Palm River	Rocky Creek
7	Delaney Creek	Rocky Creek
8	Alafia River Upstream of Lithia Gage	Lithia
8	Alafia River Downstream of Lithia Gage	Little Manatee
12	Little Manatee River	Little Manatee
9-11,13-16	Other Eastern Shore Tributaries	Little Manatee
17-33	Western Shore Tributaries	Rocky Creek
34	Lake Tarpon Canal	Rocky Creek
35	Double Branch Creek	Rocky Creek
36	Rocky Creek	Rocky Creek
37 & 38	Sweetwater W & E	Rocky Creek

*See Figure 2

TABLE 6
LOADING FACTORS USED FOR URBAN LANDUSES

Land Use (Test Basin)	Mean Concentration (mg/l)						
	Total P	Ortho-P	Total N	TKN	Nitrate-N ¹	Ammonia-N	BOD ₅
Single Family Residential							
(CH Only) ²	0.39	0.19	2.36	1.70	0.66	0.17	11.7
(CH + WD) ³	0.29	0.11	1.87	1.34	0.53	0.20	14.3
Multi-Family Residential (J.L. Young)	0.33	0.16	1.65	1.34	0.31	0.23	14.8
Commercial (Norma Park)	0.15	0.08	1.18	0.83	0.35	0.32	10.9
Industrial (Norma Park)	0.15	0.08	1.18	0.83	0.35	0.32	10.9
Institutional (N. Jesuit)	0.20	0.07	1.77	1.39	0.38	0.27	14.5
Recreational and Open Space (Rocky Creek)	0.21	0.18	1.21	1.02	0.18	0.18	1.4

1 Nitrate-N represents the sum of Nitrite and Nitrate

2 Mean concentrations based on only the Charter and Harding dataset

3 Mean concentrations based on the pooled Charter and Harding and Wilder Ditch datasets

- 1) one-sided K-S test (comparing the distributions of the two data sets);
- 2) F-test (comparing the variances of the data sets); and 3) t-test (comparing the means of the data sets).

Load Projections. In order to highlight some of the implications of the selected approach to developing flow and loading inputs, load calculations for total nitrogen, total phosphorus and BOD₅ are presented in Tables 7 through 9, respectively. In addition to low flow and high flow inputs for three different duration-frequency combinations and a thunderstorm scenario, the Tables also show average annual loading inputs based upon the total annual load for a year of average streamflow conditions. Three input points were chosen to represent the impacts due to different land use characteristics. The Hillsborough River (point 3 in Figure 2) is a 650 sq. mi. basin with a large rural/agricultural area in its upper basin and a main stem reservoir in the lower portion of the basin. The Little Manatee River (point 12) is a 149 sq. mi. basin that is almost entirely rural/agricultural. St. Petersburg Airport (point 26) is highly urbanized 17 sq. mi. basin.

The most important result to note is the difference between the 7Q2 scenarios (which represent average conditions associated with 50 percent probability of occurrence) and the thunderstorm scenario. No impact is seen at all from the Little Manatee River as it has no urban areas adjoining the Bay. Thunderstorm loads from the highly urbanized St. Petersburg Airport basin are more than double the 7Q2 loads. The Hillsborough River thunderstorm loads are much less than the 7Q2 loads because the reservoir holds back any upstream influence; therefore, urban storm runoff is generated from below the reservoir only.

Another point to note is the difference between the average annual loads and the high flow loads. All high flow loads are much greater than the average annual loads, emphasizing the importance of specifying a design duration of steady-state conditions for wasteload allocation studies. A wasteload allocation study using average annual loads could significantly underestimate the impact from nonpoint source loads.

The constituent loads from different land uses can also be analyzed in these tables. Consider the ratios of the 7Q2 loads to the thunderstorm loads for the Hillsborough River. Recalling that the reservoir collects the upstream flow (which is primarily rural/agricultural) during the thunderstorm, the thunderstorm loads are only from the highly urbanized area downstream of the reservoir. The 7Q2 loads consist of a high proportion of the rural/agricultural loadings. The ratios of these loads give an indication of how much load is attributable to the urban land uses versus the rural/agricultural land uses. The nitrogen ratio is 17.7, the phosphorus ratio is 35.0, and the BOD₅ ratio is 4.2. The phosphorus ratio reflects the high natural phosphorus content in the nonpoint source runoff. The BOD₅ load, on the other hand, is significantly influenced by the urban areas.

TABLE 7
CALCULATED LOADS
TOTAL NITROGEN
(lbs/day)

<u>Scenario</u>	<u>Hillsborough River</u>	<u>Little Manatee River</u>	<u>St. Petersburg Airport</u>
Low Flows*			
7Q10	146	19	0.73
14Q10	146	28	6.3
30Q10	152	37	7.0
High Flows*			
7Q2	35,170	10,140	1,020
15Q2	27,160	6,800	710
30Q2	19,970	4,860	497
Thunderstorm	1,990	0	2,680
Average Annual	5,180	1,140	213

* Inputs for "n-day/1-year" conditions (i.e., 7Q10 refers to 7-day/10-year streamflow).

DISCUSSION

The actual flow and load calculations presented here are simple and straight-forward. Similar data bases are available for areas all over the U.S. The general public has easy access to this data. These calculations, therefore, can be used for any wasteload allocation study.

However, the assumptions we made to allow us to make these simple calculations required a thorough investigation of the basin characteristics, both

TABLE 8
CALCULATED LOADS
TOTAL PHOSPHORUS
(lbs/day)

<u>Scenario</u>	<u>Hillsborough River</u>	<u>Little Manatee River</u>	<u>St. Petersburg Airport</u>
Low Flows*			
7Q10	13	8.0	0.12
14Q10	13	11	1.0
30Q10	15	15	1.2
High Flows*			
7Q2	11,980	4,970	177
15Q2	9,260	3,340	124
30Q2	6,810	2,380	87
Thunderstorm	342	0	447
Average Annual	1,490	520	36

* Inputs for "n-day/1-year" conditions (i.e., 7Q10 refers to 7-day/10-year streamflow).

in terms of flows and loading factors. It is here that the engineer would have to exercise considerable judgment in carrying out the analysis.

Nearfield and farfield assessments are critical in determining the proper steady-state condition to use for estuary modeling. The examples presented here show the impact of the selected design condition on the loads generated within the Tampa Bay area. This approach can be used in other waste-load allocation studies which depend upon steady-state models.

TABLE 9
CALCULATED LOADS
BOD₅
(lbs/day)

<u>Scenario</u>	<u>Hillsborough River</u>	<u>Little Manatee River</u>	<u>St. Petersburg Airport</u>
Low Flows*			
7Q10	82	36	1.3
14Q10	83	53	11
30Q10	94	70	13
High Flows*			
7Q2	63,190	19,990	6,740
15Q2	45,530	13,410	4,710
30Q2	33,470	9,580	3,300
Thunderstorm	15,170	0	19,980
Average Annual	8,360	2,320	1,010

* Inputs for "n-day/i-year" conditions (i.e., 7Q10 refers to 7-day/10-year streamflow).

REFERENCES

- (1) Ross, B.E. and P.D. Jenkins, "Computer Simulation of Nutrients in Tampa Bay, Florida," Structures, Materials, and Fluids Department, College of Engineering, University of South Florida, Tampa, FL, August 1978.
- (2) Hartigan, J.P. and S.A. Hanson-Walton, "Tributary Streamflows and Pollutants Loadings Delivered to Tampa Bay," Camp Dresser & McKee Inc., Annandale, VA, January 1984.
- (3) Hartigan, J.P., et al., "Calibration of Urban Nonpoint Pollution Loadings Models," Proceedings of ASCE Hydraulics Division Specialty Conference on Verification of Mathematical and Physical Models in Hydraulic Engineering, American Society of Civil Engineers, New York, NY, August 1978, pp. 363-372.
- (4) Priede-Sedgwick, Inc., "Runoff Characterization: Water Quality and Flow," prepared for Tampa Nationwide Urban Runoff Program Study-Phase II, March 1983.
- (5) Metcalf and Eddy, Inc., "Tampa Nationwide Urban Runoff Program (Phase II): Tasks II.6, II.9, and II.10," prepared for City of Tampa, FL, July 1983 (Draft).
- (6) U.S. Geological Survey, "Tampa Bay Region Urban Watershed Study Database: 1972-1980."
- (7) Lopez, M.A. and D.M. Michaelis, "Hydrologic Data from Urban Watersheds in the Tampa Bay Area, Florida," Water Resources Investigations Open-File Report 78-125, U.S. Geological Survey, Tallahassee, FL, 1979.
- (8) Giovanelli, R.F. and J.B. Murdoch, "Urban Stormwater Runoff in the Tampa Bay Area," prepared for Bay Area Scientific Information Symposium (BASIS), Tampa, FL, 1982.
- (9) Lopez, M.A. and W.M. Woodham, "Magnitude and Frequency of Flooding on Small Urban Watersheds in the Tampa Bay Area, West-Central Florida," Water Resources Investigations Open-File Report 82-42, U.S. Geological Survey, Tallahassee, FL, 1983.

WATER SCREEN - A MICROCOMPUTER PROGRAM FOR ESTIMATING
NUTRIENT AND POLLUTANT LOADINGS

by

Bruce L. Bird⁺ and K. Marlene Conaway⁺⁺

INTRODUCTION

Because of the high cost of carrying out significant spatial and temporal sampling to adequately characterize even a small watershed, other alternatives have been sought. A great deal of effort has been devoted to the development of detailed computer models that can be used to simulate the water, nutrient, and pollutant transport within a watershed. These detailed models include processes with short time scales so that the hydrology can follow effects of a single storm event or follow variations in daily rainfall. The models then relate sediment, nutrient, and pollutant transport to the hydrologic simulation.¹⁻⁵

Because many local agencies do not have the resources required for proper application of the detailed models, a further simplification has been made. Watersheds with sufficient data to calibrate and verify a model are used to generate average loading factors for nutrients and pollutants and for different land uses.⁶⁻⁸ A loading factor gives the average amount of pollutant produced by a given land use in terms of pounds per acre per year. If the watershed from which the loading factors are derived is similar to the watershed of local interest, then an estimate of nutrient and pollutant loadings can be made once the land use patterns of the local watershed are determined. This is one of the approaches used in the microcomputer program WATER SCREEN.

An alternative method for estimating nutrient and pollutant loadings is based on the Modified Universal Soil Loss Equation (MUSLE). The USLE was developed by soil scientists to estimate soil loss from agricultural fields. It was later modified by the addition of a sediment delivery factor so that the amount of soil that reaches a natural water body can be estimated. Nutrient and pollutant loadings can be related to the amount of delivered sediment.⁹⁻¹¹ This approach is also used in WATER SCREEN to allow comparison with the loading factor method for forest and agricultural land use.

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WATER SCREEN

A computer program, WATER SCREEN, has been written for the Apple II microcomputer.¹² This program can be used by land use planners to estimate the amounts of sediment, nitrogen, phosphorous, BOD, lead, and zinc that are produced by various land use patterns within a watershed. Both a modified universal soil loss equation and a loading factor method are used in the program to estimate nutrient and pollutant loadings.

The computer program WATER SCREEN uses input parameters provided by the operator to calculate nutrient and pollutant loadings from a watershed. In the first part of this report, personnel requirements, computer hardware, program logic, and loading equations for this program are described. Application of WATER SCREEN to a small watershed is described in the second part of this report.

Personnel Requirements

A person with minimum computer experience can run WATER SCREEN. A knowledge of BASIC is not required; however, some acquaintance with BASIC on the level of Apple II User's Guide, by Poole, would allow the operator to take full advantage of the logical structure and data file generating capabilities of the program. The user need not be an expert in hydrology or ecology, but he/she should become familiar with the material covered in references 6 and 9.

Machine Requirements

WATER SCREEN has been written and run on an Apple II microcomputer system with 48K of memory. The system consists of a single disk drive and a NEC PC-8023A printer connected to the Apple II using the Orange Micro Grappler interface. As written, for seven subwatersheds, the program without REMARK statements occupies 29K of memory. It is estimated that a watershed with up to 20 subwatersheds could be run before one would run out of memory on a 48K machine. If more subwatersheds were needed, the watershed could be separated into smaller sections and then each section evaluated using the program. While the program is written in Applesoft BASIC, it could readily be modified for use on other machines using the appropriate translations. The printer commands are listed in separate subroutines and can be easily modified for other printers if needed. Modifications of the file commands used to store input data on disk would require the most effort.

Program Logic

WATER SCREEN can easily be modified to accommodate changes that may be proposed to the algorithms now used or to incorporate new algorithms. The program consists of three major sections: (1) create, read, and edit input data file; (2) calculation of pollutant and nutrient loading using the modified universal soil loss equation (MUSLE) and algorithms described by Zison et al⁹ for nitrogen, phosphorous and organic matter; (3) calculation of nutrient and pollutant loading using loading factors developed by the Northern Virginia Planning District Commission (NVPDC) in their extensive study of the Occoquan River Basin.⁶

The data file part of the program asks the user to input factors for the MUSLE and for soil and rainfall information needed to calculate nitrogen, phosphorous, and organic loadings. Loading factors for various categories of land use must also be input to the program. Much of this information is requested by subwatershed so that variations within the watershed can be accommodated.

The application of WATER SCREEN to the Church Creek watershed will be discussed in later sections. For this watershed with six subwatersheds it takes about twenty minutes to input all the required data. The program stores this data in a disk file where it can later be recalled and edited if desired. The run time of the program depends upon the number of subwatersheds selected and the speed of the printer. The printout of the results from the Church Creek watershed takes about twenty minutes using an eighty characters per second printer. Subtotals and totals are printed by subwatershed, land use, and pollutant type.

The file editing features makes it possible to quickly see the sensitivity of the calculation to changes in the value of a parameter, such as slope or loading factor, by running the program, using the editing feature to change the desired parameter, and then rerunning the program.

A flow diagram for the major subsections of WATER SCREEN is shown in Figure 1a and 1b. Because of the logic used in the program it is quite easy to add or delete subroutines that use different methods of calculating nutrient and pollutant loadings and to add or delete the corresponding records that contain the parameters used in the calculation. Commercially available utility programs, such as Apple Doc, are very helpful in this process.

Algorithms Used in WATER SCREEN

A. Modified Universal Soil Loss Equation

Agricultural and soil scientists have been concerned for decades with the problem of controlling soil loss from farm land. The universal soil loss equation was developed to estimate soil loss from a field and to predict the effects of crop rotation, management practices, etc. The USLE has the form:

$$Y(S)_E = A R K L S C P$$

where

$Y(S)_E$ = soil loss due to surface erosion (tons/year)

A = area of field (acres)

R = rainfall factor, indicates the erosion potential of average annual rainfall (R unit)

K = soil-erodibility factor (tons/acre/R unit)

L = slope-length factor (dimensionless ratio)

S = slope-steepness factor (dimensionless ratio)

C = cover factor (dimensionless ratio)

P = erosion control practices factor (dimensionless ratio)

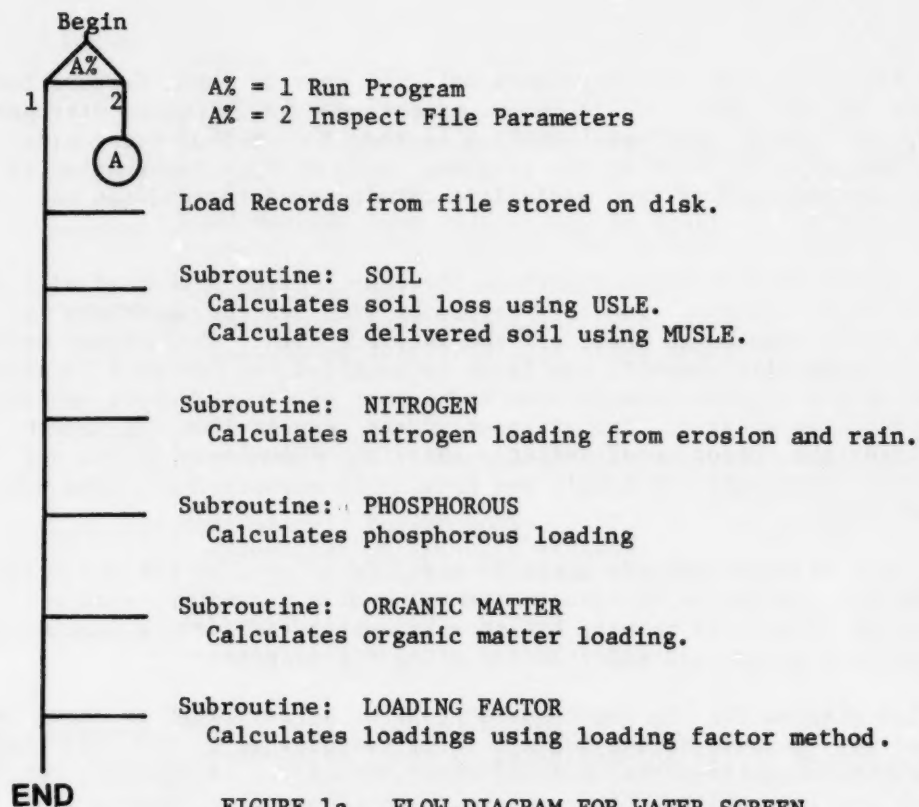


FIGURE 1a FLOW DIAGRAM FOR WATER SCREEN

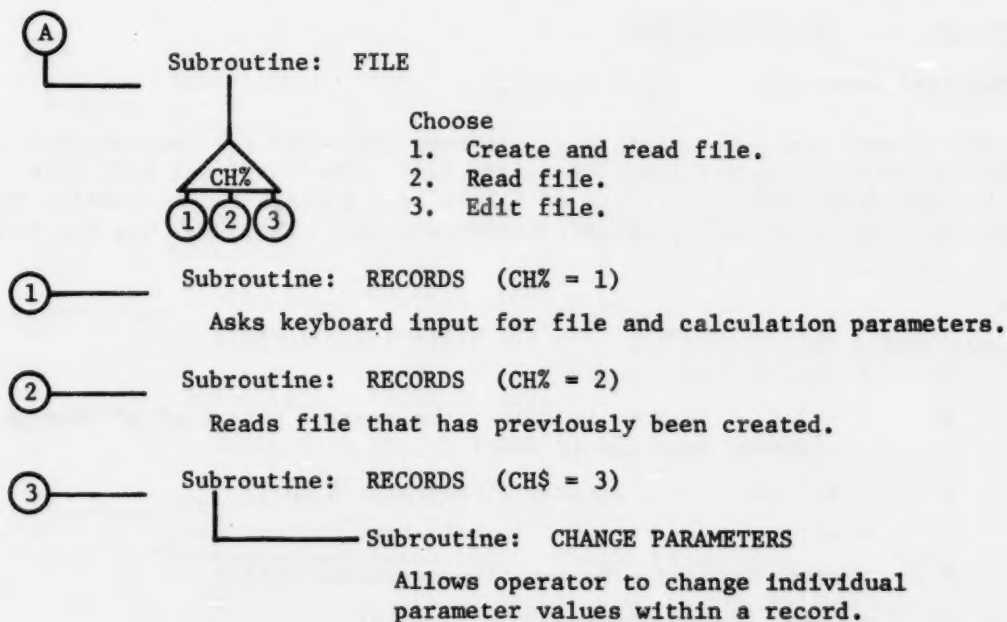


FIGURE 1b FLOW DIAGRAM FOR WATER SCREEN

The U.S. Department of Agriculture has carried out extensive experimental studies on fields located throughout the United States. From these studies accepted values for the factors in the USLE have been tabulated. Local values are obtainable from handbooks published by the Soil Conservation Service.¹⁴ As has been pointed out by various authors, the USLE only predicts the soil lost from a field. It does not estimate the actual amount of soil that is delivered to a stream channel. In order to estimate this, an additional factor, the sediment delivery ratio, is needed.¹⁵

To estimate the amount of soil delivered to a stream the modified universal soil loss equation (MUSLE) is used. The MUSLE has the form:

$$Y(S)_D = Y(S)_E S_D$$

where

$Y(S)_D$ = soil delivered to stream channel (tons/year)

$Y(S)_E$ = soil lost from field (tons/year)

S_D = sediment delivery ratio (dimensionless ratio)

The sediment delivery ratio is probably the most difficult factor in the MUSLE to estimate with any degree of reliability.¹⁵ Its value in different watersheds has been obtained by observing the silting rate of dams. Obtained this way it accounts not only for field losses but also all other erosion processes, such as stream channel erosion, which are not accounted for by the USLE.

It is generally accepted that the soil lost from an individual field can be reliably estimated using appropriate local values for the factors in the USLE. However, it is quite another matter to reliably estimate the sediment loading from a watershed composed of many fields with varying slopes, ground cover, distance from stream channel, etc. For example, it is not clear how to handle fields with concave slopes. Soil lost from one part of a field may redeposit in another part of the same field or in another part of the watershed.¹⁶

The MUSLE has been applied primarily to agricultural and forest land although some work has been done to extend it to residential land use. In this report the MUSLE will only be applied to forest and agricultural land use.

B. Nitrogen Loading Function

Zison et al⁹ point out that the movement of nitrogen compounds within an ecosystem is complex and still not thoroughly understood. For an estimate of the amount of total nitrogen produced by runoff and erosion, excluding leaching losses, they suggest the following expression:

$$Y(NA) = f_N Y(NT)_E + Y(N)_{Pr}$$

where

$Y(NA)$ = total available nitrogen (lbs/year)

f_N = ratio of available to total nitrogen in sediment (dimensionless)

$Y(NT)_E$ = total nitrogen loading from erosion (lbs/year)

$Y(N)_{Pr}$ = stream nitrogen loading from precipitation (lbs/year)

$Y(NT)_E$ is found from the expression:

$$Y(NT)_E = 20 Y(S)_D C_S(NT) r_N$$

where

$Y(S)_D$ = soil delivered to stream channel (tons/year)

$C_S(NT)$ = total nitrogen concentration in soil (g/100g)

r_N = nitrogen enrichment ratio

The factor of 20 takes into account the units used in this equation.

$Y(N)_{Pr}$ is found from the expression:

$$Y(N)_{Pr} = A \frac{Q(OR)}{Q(Pr)} N_{Pr} b$$

where

A = area (acres)

Q(OR) = overland flow from precipitation (in/year)

Q(Pr) = total amount of precipitation (in/year)

N_{Pr} = nitrogen loading in precipitation (lb/acre/year)

b = attenuation factor

Zison et al⁹ discuss the methods of evaluation of the parameters in the nitrogen loading function.

C. Phosphorous Loading Function

A great deal of confusion still exists about the proper terms for describing the various physical and chemical states of phosphorous. There is also some disagreement about the effectiveness of chemical extraction procedures to selectively remove a particular phosphate from the soil.¹⁷ Because of these uncertainties the values of the parameters used in the loading function for phosphorous are only rough approximations. Zison et al⁹ suggest that based on the soil erosion transport mechanism the loading function for phosphorous should have the form:

$$Y(PA) = 20 f_P Y(S)_D C_S(PT) r_P$$

where

$Y(PA)$ = loading of available phosphorous (lbs/year)

f_P = ratio of available phosphorous to total phosphorous (dimensionless)

$Y(S)_D$ = soil delivered to stream channel (tons/year)

$C_S(\text{PT})$ = total phosphorous concentration in soil (g/100g)

r_P = phosphorous enrichment ratio

Zison et al.⁹ discuss the methods for determining the parameters in the phosphorous loading function.

D. Organic Matter Loading Function

Zison et al.⁹ also suggest a loading function for organic matter of the form:

$$Y(\text{OM})_E = 20 C_S(\text{OM}) Y(S)_D r_{\text{OM}}$$

where

$Y(\text{OM})_E$ = organic loading (lbs/year)

$C_S(\text{OM})$ = organic matter concentration in soil (g/100g)

$Y(S)_D$ = soil delivered to stream channel (tons/year)

r_{OM} = enrichment ratio for organic matter in eroded soil

Procedures for determining the values of the parameters in the organic loading function are given by Zison et al.⁹

E. Loading Factors

During the last seven years field and detailed modeling studies of non-point pollution from small watersheds with one predominant land use have been funded by the Water Resources Planning Board, Washington Council of Governments, and by the Environmental Protection Agency Chesapeake Bay Program. The modeling studies were carried out by the Northern Virginia Planning District Commission (NVPDC). The earlier watershed studies on the Occoquan River Basin were done by personnel from the Civil Engineering Department, Virginia Polytechnic Institute. Later watershed measurements were done in the Ware River Basin (southeastern Virginia), Pequea Creek Basin (Lancaster, Pennsylvania), Patuxent River Basin (western shore of Chesapeake Bay), and Chester River Basin (eastern shore of Chesapeake Bay). These measurements were done by groups from Virginia Institute of Marine Science, U.S. Geological Survey, and State of Maryland, respectively.¹⁸⁻²²

As a result of this work non-point pollution loading factors have been generated which can be used to estimate loadings produced in a watershed by using the equation:

$$Y(X) = \sum_{i=1}^N F_i(X) A_i$$

where

$Y(X)$ = loading of pollutant X (lbs/year)

$F_i(X)$ = loading factor for pollutant X and land use i (lbs/acre/year)

A_i = area of land use i (acres)

In this report we have used the loading factors given in reference 6. A recalibration of these factors has recently been reported.⁸

APPLICATION OF WATER SCREEN TO THE CHURCH CREEK WATERSHED

Introduction

A small watershed was selected to demonstrate the application of the WATER SCREEN program. In this section we describe the Church Creek watershed land use patterns of this watershed in 1974, 1981, and a hypothetical development pattern based on the General Development Plan (GDP), and the choice of input parameters for WATER SCREEN. We then compare the results of the modified universal soil loss equation (MUSLE) method with the loading factor approach for forest and agricultural land use, and discuss the estimated loadings for all forest, 1974, 1981, and GDP land use patterns.

Description

Church Creek is located just south of Annapolis, Maryland. It is a mile-long tidal tributary of South River, which is a subestuary of Chesapeake Bay. The watershed has an area of 1200 acres. The soil type within the watershed is predominantly Monmouth loamy sand with some Collington fine sandy loam. Typical land slopes are in the 0 to 10% range with higher slopes on land close to the shores of the Creek.

For purposes of analysis the watershed was divided into six sub-watersheds based on water drainage patterns. The sub-watershed boundaries are indicated by dashed lines in Figures 2, 3 and 4, and are identified by SW1, SW2, etc. The land use patterns that existed in 1974 and 1981 are shown in Figures 2 and 3, using the code shown in Table 1. A hypothetical land use pattern based upon the present General Development Plan is shown in Figure 4. It should be emphasized that this projected land use is not possible because of existing development patterns and regulatory controls. It is presented as an extreme to which present land use can be compared. The relative percentage of land use for 1974, 1981 and the GDP is shown in Figure 5. The land use classifications used in this report are forest, pasture, hayfield, conventional tillage crop, minimum tillage crop, idle, low density residential ($\frac{1}{2}$ -1 dwelling unit [DU] per acre), low/medium density (2-5 DU/acre), medium density (5-10 DU/acre), high density (greater than 10 DU/acre), and commercial.

TABLE 1 LAND USE CODES

1 Forest	7 Low Density Residential
2 Pasture	8 Low/Medium Density Residential
3 Hayfield	9 Medium Density Residential
4 Conventional Crop	10 High Density Residential
5 Minimum Tillage	11 Commercial
6 Idle	

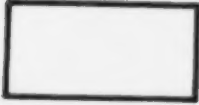
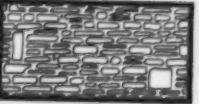
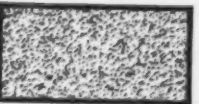
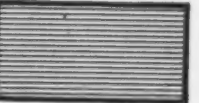
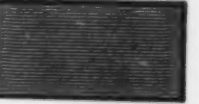
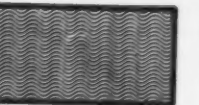
	FOREST		LOW/MEDIUM DENSITY 2-5 DU/ACRE
	PASTURE		MEDIUM DENSITY 5-10 DU/ACRE
	HAYFIELD		HIGH DENSITY 10 OR MORE DU/ACRE
	CROPLAND		COMMERCIAL- GENERAL
	MINIMUM TILLAGE		COMMERCIAL UNDER CONSTRUCTION
	IDLE		INDUSTRIAL PARK
	LOW DENSITY 1/2-1 DU/ACRE		

Table 1 (continued)
Land Use Code

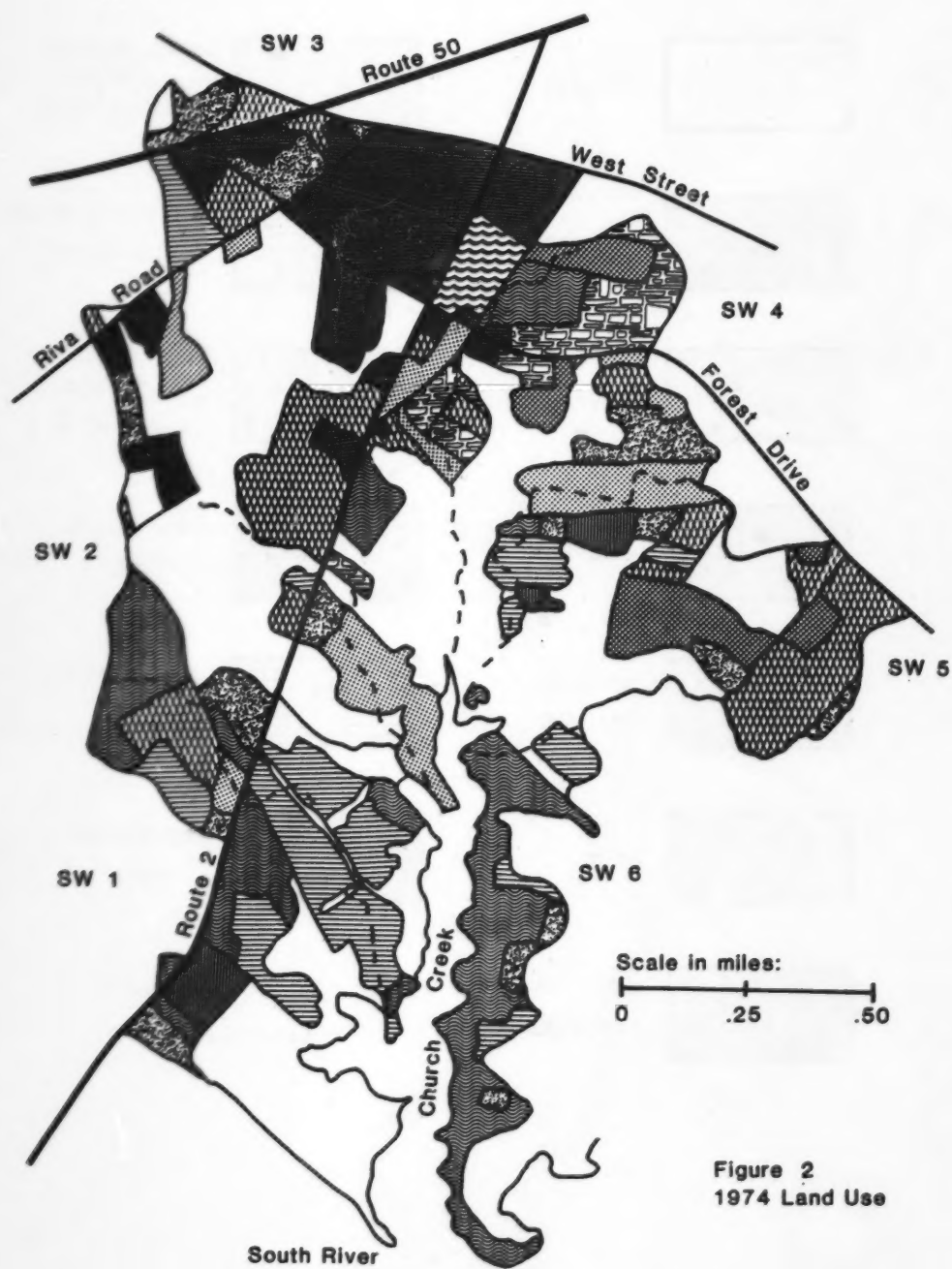
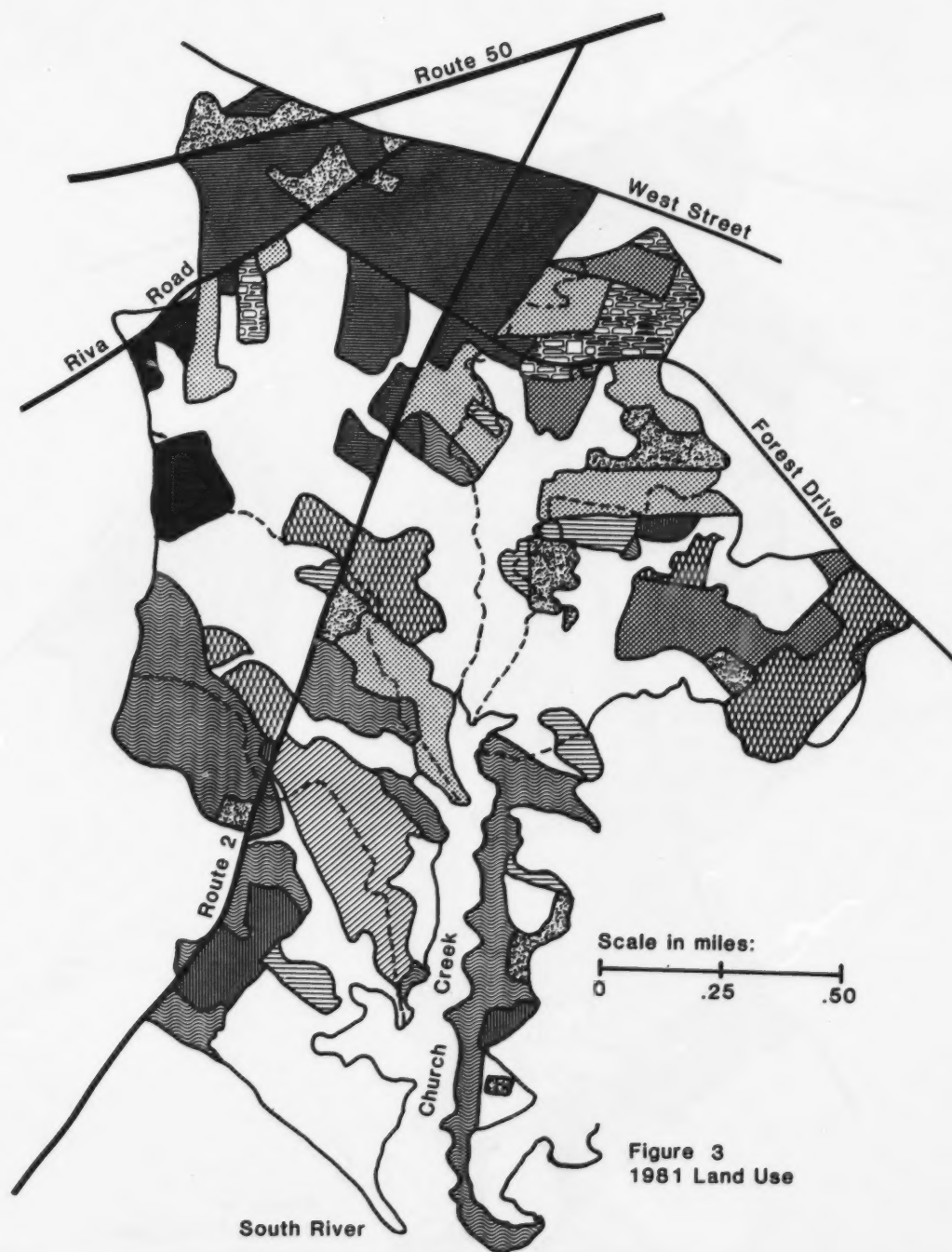


Figure 2
1974 Land Use



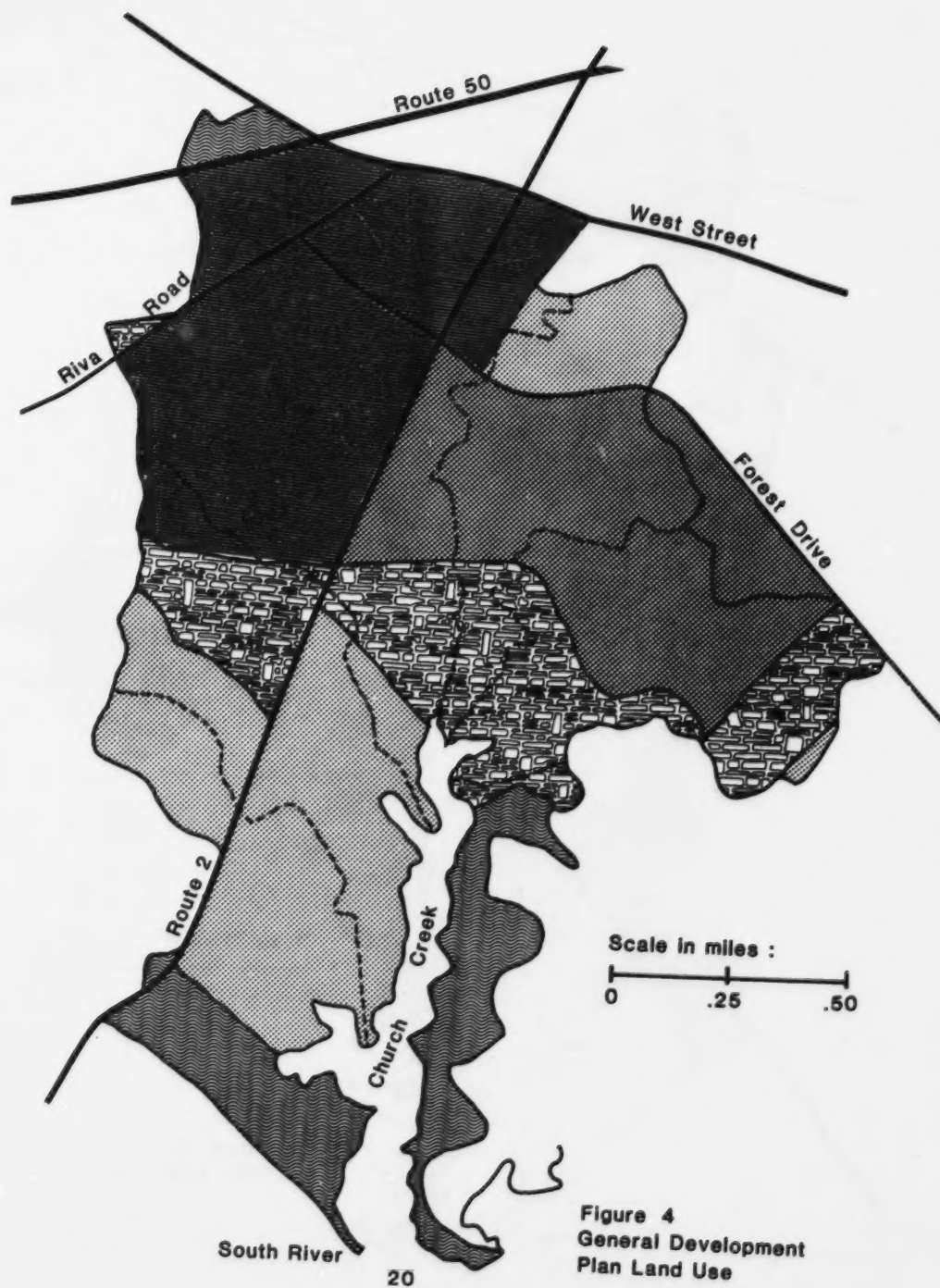


Figure 4
General Development
Plan Land Use

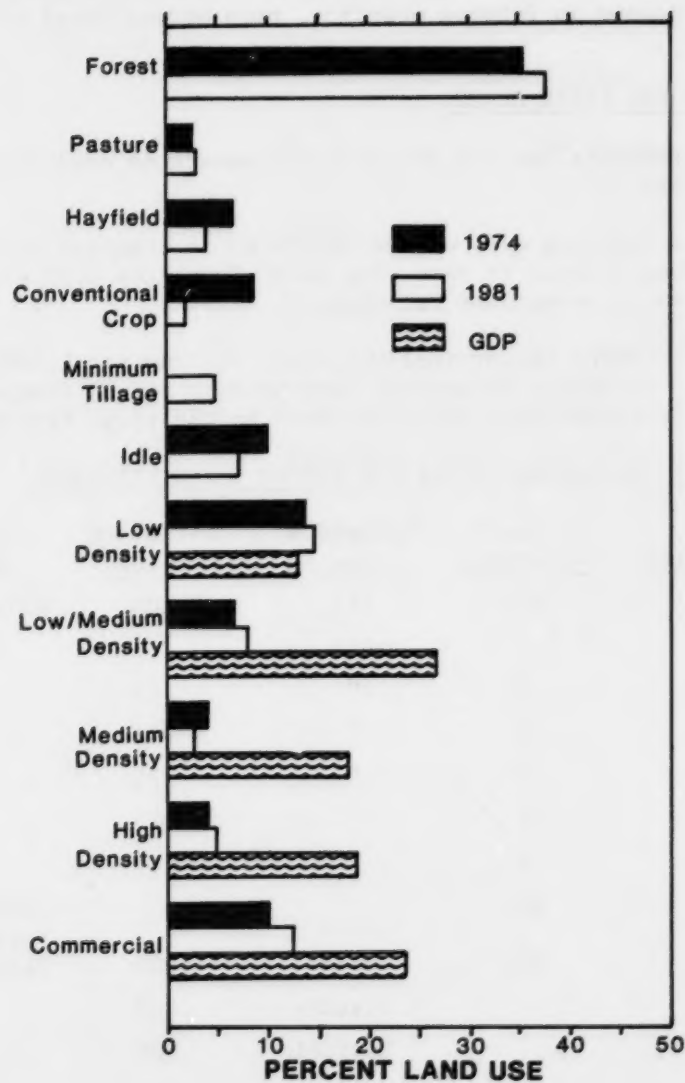


Figure 5 Percentage of Land Use
in Church Creek Watershed

Commercial areas are located primarily in subwatersheds 3 and 4. These are shopping centers or office buildings. There is no heavy industrial land use within the watershed. Agricultural land lies primarily in subwatersheds 1, 2 and 6. Comparison of 1974 with 1981 land use indicates an increase of commercial land by 25% and a decrease in conventional tillage land by 78% largely due to changes in farming practice, from conventional to minimum tillage.

Input Parameters for WATER SCREEN

Input parameters for the Church Creek watershed used in WATER SCREEN are listed in Table 2.

Soil maps indicate most of the watershed is composed of loamy sand, but in subwatershed 4 there is some fine sandy loam; the soil erodibility factor K was adjusted to account for this.

The length-slope factor (LS) was obtained from the 1:4800 contour maps by "eyeball" estimates of average lengths to a stream channel and typical changes in elevation. LS values were in the range from 0.2 to 3.0.

TABLE 2 PARAMETER VALUES FOR CHURCH CREEK WATERSHED

<u>Function</u>	<u>Parameter</u>	<u>Sub- Watershed</u>	<u>Land Use</u>	<u>Parameter Value</u>	<u>Source</u>
MUSLE	R	A11	A11	200	Reference 9
	K	1	A11	.43	" "
		2	"	.43	
		3	"	.43	
		4	"	.35	
		5	"	.43	
		6	"	.43	
"	LS	A11			1/4800 scale maps Reference 9
"	C	A11	Forest	.003	Reference 14
			Pasture	.013	
			Hayfield	.009	
			Idle	.012	
			Conventional Crop	.319	
			Minimum Tillage Crop	.185	
"	P	A11	Non-cropland	1.0	Reference 14
			Cropland	.75	

TABLE 2 PARAMETER VALUES FOR CHURCH CREEK WATERSHED (CONT'D)

<u>Function</u>	<u>Parameter</u>	<u>Sub- Watershed</u>	<u>Land Use</u>	<u>Parameter Value</u>	<u>Source</u>
MUSLE	S_D	1	All	.5	1/4800 scale maps
		2	"	.54	Reference 9
		3	"	.48	
		4	"	.54	
		5	"	.54	
		6	"	.56	
NITROGEN	f_N	All	Forest & Agriculture	.02	Local Value
	$C_S(NT)$	"	"	.07 g/100g	Reference 9
	r_N	"	"	2.1	Local Value
	$Q(OR)$	"	"	28.8 in/year	Reference 9
	$Q(PR)$	"	"	48 in/year	Local Value
	N_{Pr}	"	"	1.78 lb/acre/yr	Reference 9
	b	"	"	.75	Reference 9
PHOPHOROUS	f_P	All	Forest &	.048	Local Value
	$C_S(PT)$	"	Agriculture	.07 g/100g	Reference 9
	r_P	"	"	1.5	Reference 9
ORGANIC MATTER	$C_S(OM)$	All	Forest & Agriculture	1.4 g/100g	Reference 9
	r_{OM}	"	"	1.5	Reference 9

Some of the loading factors listed in Table 3 have been revised based on recent recalibration measurements.

Comparison of MUSLE method with Loading Factor Approach

Loading estimates from both the MUSLE and loading factor methods are shown for sediment, total nitrogen, and total phosphorous in Figures 6, 7 and 8 respectively. These results are only for forest and agricultural land use within the watershed. As can be seen, the MUSLE method estimates a much larger amount of sediment, by a factor of 10 to 20, than that estimated from the loading factor method. However, estimates for total nitrogen and total

TABLE 3 LOADING FACTORS (LBS/ACRE/YEAR) SELECTED FOR CHURCH CREEK WATERSHED

<u>Pollutant</u>	<u>Land Use</u>					
	<u>Forest</u>	<u>Pasture</u>	<u>Hayfield</u>	<u>Conventional Crop</u>	<u>Minimum Tillage</u>	<u>Idle</u>
Sediment	20	20	20	1580	900	20
Nitrogen	2.4	4.3	2.6	12.5	8.7	2.6
Phosphorous	.1	.3	.1	2.3	1.1	.1
BOD	6	13	6	29	19	6
Lead	.01	.01	.01	.02	.02	.01
Zinc	.01	.01	.01	.1	.1	.01

<u>Pollutant</u>	<u>Land Use</u>				
	<u>Low Density</u>	<u>Low/Medium Density</u>	<u>Medium Density</u>	<u>High Density</u>	<u>Commercial</u>
Sediment	120	240	420	560	480
Nitrogen	5.1	7.1	9.7	12.5	13.2
Phosphorous	.5	.8	1.1	1.7	1.6
BOD	13	18	25	36	163
Lead	.12	.29	.59	.97	2.58
Zinc	.11	.23	.38	.55	2.06

phosphorous are just the opposite; the MUSLE estimates are a factor of 4 to 15 times lower than those given by the loading factor approach. Only in the case of minimum tillage crop are the estimates of comparable size. Both the nitrogen and phosphorous loading functions used with the MUSLE approach relate the amount of nutrient to the amount of sediment; therefore one would expect the sediment, nitrogen, and phosphorous estimates to all be high or low, but they are not.

Possible reasons for the discrepancy between the two methods are:

1. Nitrogen and phosphorous loading functions that were used are not correct or not appropriate for this watershed.
2. The loading factors used are not appropriate for this watershed.

The transport of nitrogen and phosphorous have been intensively studied because of their importance in the eutrophication of natural waters.^{17,24,25}

They are part of an extremely complex process with some of the details still not well understood. It is therefore not unlikely that the simplified method of estimating nitrogen and phosphorous loadings by means of loading functions is not correct.

Recent work in the Chesapeake Bay Program found that the loading factors developed for the Occoquan River basin in Northern Virginia were comparable to values found in small watersheds located in the Pequea Creek region near Lancaster, Pennsylvania.²² Work with other watersheds within the Chesapeake Bay watershed also indicated comparable loading factors. This suggests that these loading factors are appropriate for use with the Church Creek watershed provided there are no unique characteristics of the Church Creek watershed which distinguish it from the watersheds on which the loading factors are based.

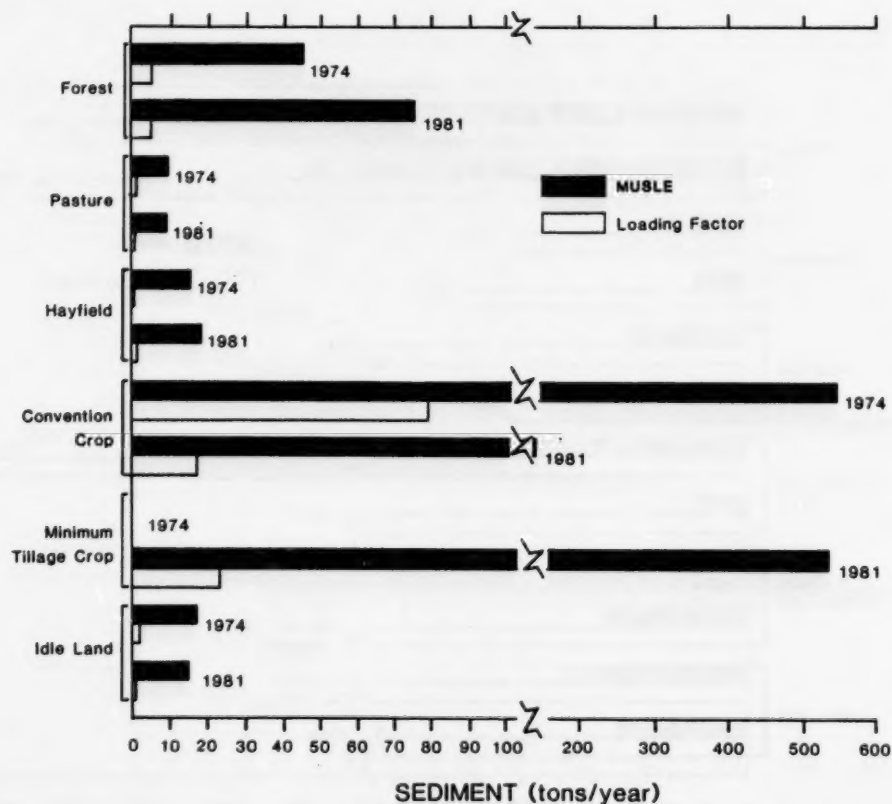


Figure 6
Comparison of MUSLE and
loading factor estimates-sediment

While writing this report we received the recently revised loading factors from NVPDC based on additional measurements.⁸ There were no changes in the nitrogen, phosphorous, and BOD loading factors. A 33% reduction in the zinc loading factor for single family residential (0.5-6.0 DU/acre) and townhouse garden apartments (6.0-20.0 DU/acre) was recommended. For lead a 50% reduction was recommended for single family residential and a 33% reduction for townhouse garden apartments. Based on a better sampling technique for suspended solids they recommend a 33% reduction in the sediment loading factor for all residential land use categories.

Part of the reduction in the lead loading factor may be due to the increased use of no-lead gasoline so that the original lead loading factor may be more appropriate for the 1974 and 1981 land use patterns. In any case, the revised loading factors do not significantly affect the total nutrient and pollutant loading estimates in this report which are based on the earlier loading factors from NVPDC.

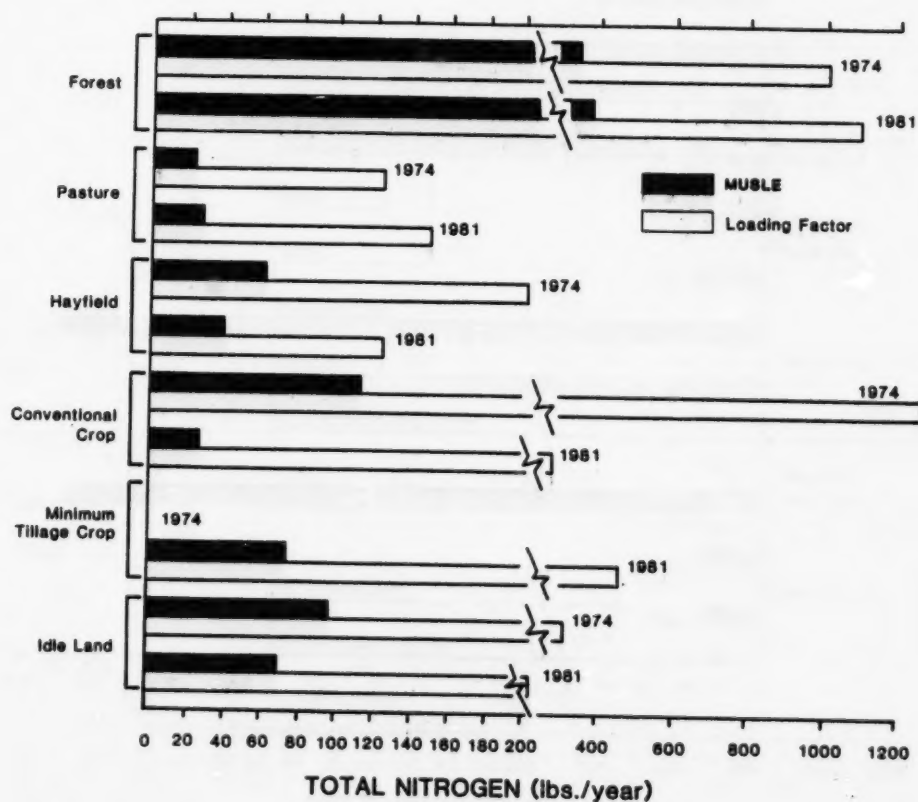


Figure 7
Comparison of MUSLE and loading factor estimates -Total nitrogen

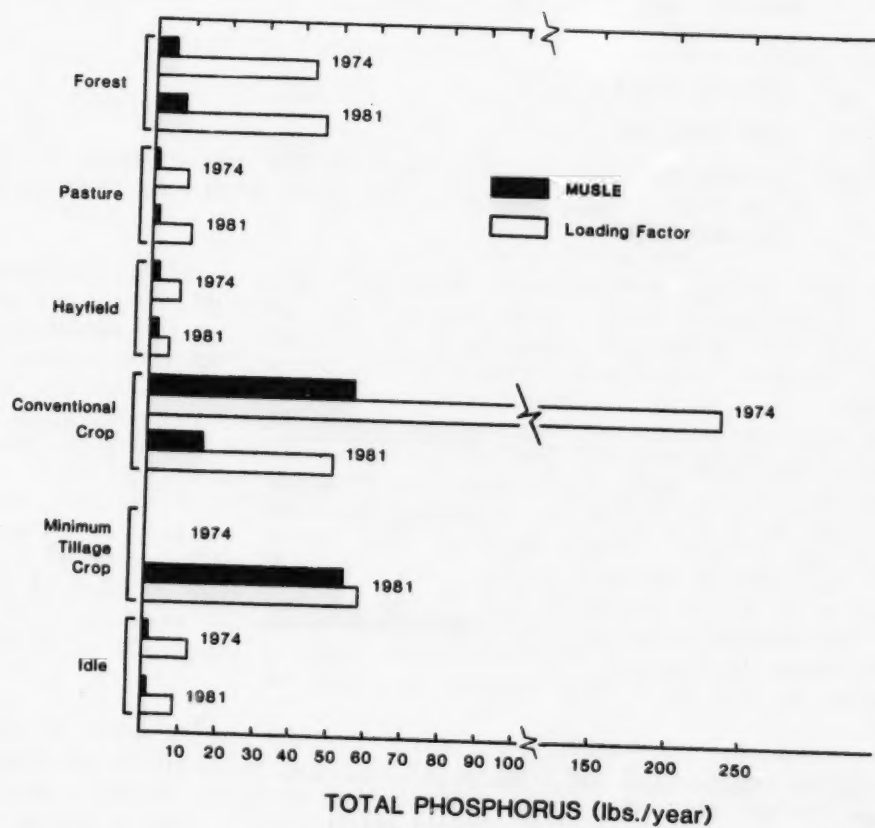


Figure 8
Comparison of MUSLE and loading
factor estimates—Total phosphorus

Nutrient and Pollutant Discharge to Church Creek

The loading factor method was used to estimate nutrient and pollutant loadings for four different land use patterns:

1. The appropriate loading factor for forest land use was multiplied by the total land area (1200 acres) to give an estimate for each pollutant of the total loadings if the whole watershed were completely forested. This is called the "all forest" land use pattern and serves as an index of minimum loadings from the watershed.
2. 1974 land use.
3. 1981 land use.
4. Hypothesized development based on the current General Development Plan (GDP). This serves as an index of the maximum loading from the watershed.

The loadings for sediment, total nitrogen, total phosphorous, biological oxygen demand (BOD), lead, and zinc for each of the eleven categories of land use were estimated. Figures 9 and 10 summarize the estimated total pollutant loading by pollutant type for the four different development patterns considered. In general, no significant change in loadings between 1974 and 1981 was noted except in two categories. The change from conventional to minimum tillage practices on some of the farmland reduced the sediment, total nitrogen, and total phosphorous from this land use by a significant amount. On the other hand, increase in commercial land use increased BOD, lead, and zinc loadings by 25%. If one uses the "all forest" values for normalization, then by 1981 yearly sediment loading had increased by a factor of 10, total nitrogen by 2, total phosphorous by 7, BOD by 4.5, lead by 60, and zinc by 40. If the hypothetical land use patterns based on the GDP were to occur, then sediment would increase by a factor of 18, total nitrogen by 4, total phosphorous by 12, BOD by 9, lead by 100, and zinc by 75.

Comparing the loadings for all four land use patterns indicates that high density residential and commercial land use can contribute significant loadings for all the pollutants studied. For example, in 1981, sediment from commercial land was almost as much as from cropland. One of the more surprising results is the large amount of BOD loading produced by commercial land. In 1981 the BOD loading from commercial land use in the watershed was a factor of three larger than would have been produced if all the land were still forested, even though commercial land in 1981 made up only 12% of the entire watershed. Loadings of zinc and lead are also disproportionately higher for commercial land as compared to other land uses within the watershed.

A process that is not accounted for in these calculations is the disturbance and exposure of soil at construction sites or new road construction. At present, there are no good procedures for estimating the nutrient and pollutant discharges from construction sites. The reason for this is the uniqueness of each construction site in terms of land disturbance, sediment control practices, intensity and duration of storms that occurred while

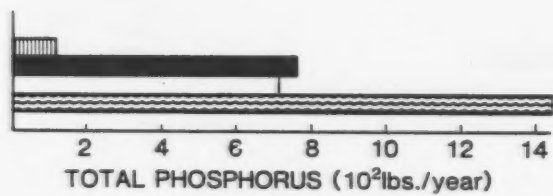
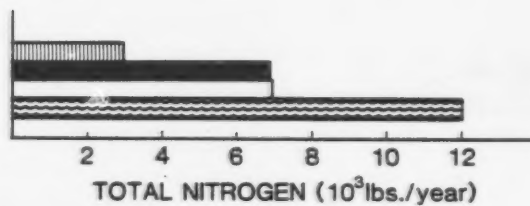
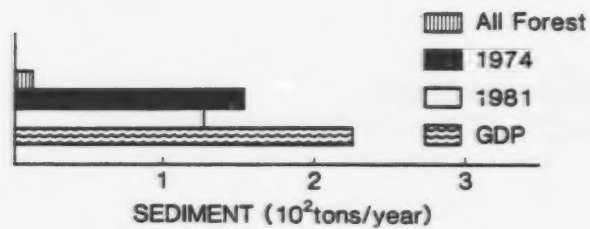


Figure 9
Total Pollutant Loading from Church Creek
Watershed (Sediment, Nitrogen, Phosphorous)

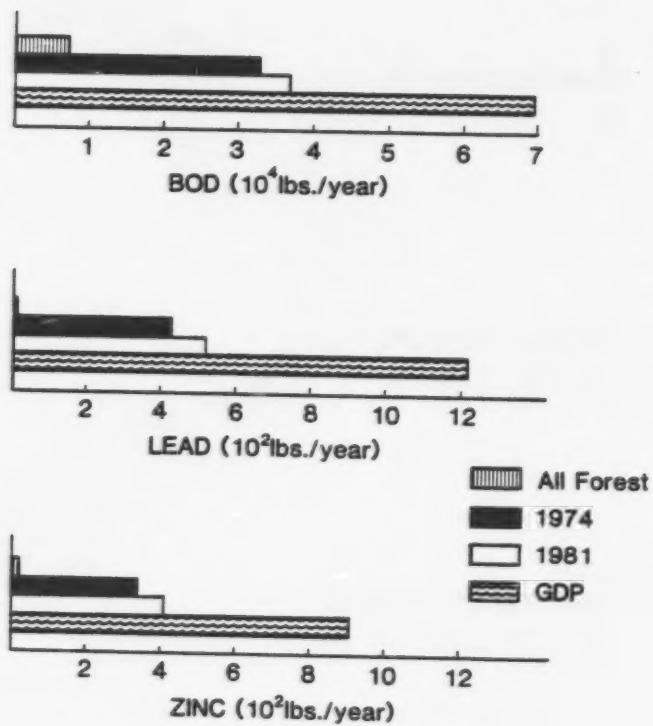


Figure 10
Total Pollutant Loading from
Church Creek Watershed (BOD,Lead, Zinc)

bare soil was exposed, etc. Because construction activity has been neglected in our model the estimated loadings are probably conservative.

Discussion

In the previous section we have obtained estimates of yearly nutrient and pollutant loadings from the Church Creek watershed. In this section we discuss the following questions:

1. What happens to these nutrients and pollutants once they enter Church Creek?
2. What impact do these nutrients and pollutants have on the water quality of Church Creek?

The first question cannot be answered without a thorough physical, chemical, and biological study of Church Creek. Even if this were available, many of the chemical and biological interactions that control the transport of nutrients and pollutants within an estuary are still not completely understood. This is particularly true of the water-sediment interface at the bottom of the creek.²⁶

The second question is not answerable at the present time. There is insufficient scientific knowledge to predict the amount of a particular nutrient or pollutant that will cause a decline in the water quality of an estuary.^{27,28} There is not even a consensus among scientists as to the proper choice of variables required to define the water quality of an estuary.²⁹⁻³¹

The impact of nutrient enrichment on an estuary and corresponding management implications are discussed in an excellent collection of review articles edited by Neilson and Cronin.³² The overall sense of these reviews is that while a great deal has been learned about nutrient cycling within an estuary, much still remains to be discovered.

With the present state of knowledge of estuarine ecology, what method can planners use to establish reasonable patterns of land use within a watershed? We suggest that the ratios R_i , where

$$R_i = \frac{\text{total loading of pollutant } i \text{ from watershed (lbs/year)}}{\text{total loading of pollutant } i \text{ for completely forested watershed (lbs/year)}}$$

be used as the index of the state of the watershed. If there is a correlation between the "health" of an estuary and these pollutant ratios, then maximum ratio values could be assigned based on comparison of several watersheds within a county. For this purpose the "health" of an estuary could be based on fairly broad criteria, such as water clarity, odor production, dissolved oxygen history, etc. Adjustments to the maximum ratios may be needed to account for factors such as flushing time and quality of benthic material.

These maximum ratio values could be used as guidelines in the establishment of general development plans for a watershed. Admittedly,

the establishment of maximum loading ratios is not a trivial task and is well beyond the scope of work reported here. We offer it as a suggestion that we believe is reasonable and within the resources of local government agencies. Planning guidelines based on watershed characteristics will focus the attention of both planners and citizens on the ecological unit that often determines the health of an estuary--its watershed.

Acknowledgement

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References

1. Swank, R.R., "U.S. Environmental Protection Agency Program On Nonpoint Source Modeling," in Environmental Impact of Nonpoint Source Pollution, M.R. Overcash and J.M. Davidson, eds., (Ann Arbor Science, 1980), p. 1.
2. Simons, D.B. and R. Li, "Modeling of Sediment Nonpoint Source Pollution from Watersheds," in Environmental Impact of Nonpoint Source Pollution, M.R. Overcash and J.M. Davidson, eds., (Ann Arbor Science, 1980), p. 341.
3. Donigan, A. and N. Crawford, "Modeling - Nonpoint Pollution from the Land Surface," EPA 600/3-76-083, EPA, Athens, GA., July 1976.
4. Johanson, R.C., J.C. Imhoff, and H. Davis, "Users Manual for Hydrological Simulation Program - FORTRAN (HSPF)," EPA 600/9-80-015, EPA, Athens, GA., April 1980.
5. Haith, D.A., "Models for Analyzing Agricultural Nonpoint Source Pollution," Research Report 82-17, International Institute for Applied Systems Analysis, April 1982.
6. Northern Virginia Planning District Commission, "Guidebook for Screening Urban Nonpoint Pollution Management Strategies," prepared for Metropolitan Washington Council of Governments, Washington, DC, November 1979.
7. Northern Virginia Planning District Commission and Virginia Polytechnic Institute and State University, "Occoquan/Four Mile Run Nonpoint Source Correlation Study," Final Report prepared for Metropolitan Washington Council of Governments, Washington, DC, July 1978.
8. Northern Virginia Planning District Commission, "Washington Metropolitan Area Urban Runoff Demonstration Project," Final Report prepared for Metropolitan Washington Council of Governments, Washington, DC, April 1983.
9. Zison, S.W., K.F. Haven, and W.B. Mills, "Water Quality Assessment - A Screening Method for Nondesignated 208 Areas," EPA 600/9-77-023, EPA 600/9-77-023, EPA, Athens, GA., 1977

10. Davis, M.J., M.K. S-yder, and J.W. Nebgen, "River Basin Validation of the Water Quality Assessment Methodology for Screening Nondesignated 208 Areas. Volume I: Nonpoint Source Load Estimation," EPA 600/3-82-057a, EPA, Athens, GA., 1982.
11. Dean, J.D., B. Hudson, and W.B. Mills, "River Basin Validation of the Water Quality Assessment Methodology for Screening Non-designated 208 Areas, Volume II: Chesapeake-Sandusky Nondesignated 208," EPA 600/3-82-057b, EPA, Athens, GA, 1982.
12. Bird, Bruce L. and K.M. Conaway, "Church Creek Watershed - An Illustration of the Application of Microcomputers For Estimating the Effects of Land Use Changes on Nutrient and Pollutant Loading," Environmental Center, Anne Arundel Community College, June 1983.
13. Wischmeier, W.H., "A rainfall erosion index for a universal soil-loss equation," Soil Sci. Soc. Amer. Proc. 23: 246-249 (1959).
14. Stephens, H., "Guide for Predicting Rainfall-Erosion Losses from Agricultural Land in Maryland and Delaware," Technical Note Conservation Planning 1-78, Soil Conservation Service, USDA, College Park, Maryland, July 1978.
15. Walling, D.E., "The Sediment Delivery Problem," J. of Hydrology, 65, (1983) 209.
16. Foster, G.R., "Soil Erosion Modeling: Special Considerations for Nonpoint Pollution Evaluation of Field Sized Areas," in Environmental Impact of Nonpoint Source Pollution, M.R. Overcash and J.M. Davidson, eds., (Ann Arbor Science, 1980), p. 213.
17. Logan, T.J., "The Role of Soil and Sediment Chemistry in Modeling Nonpoint Sources of Phosphorous," in Environmental Impact of Nonpoint Source Pollution, M.R. Overcash and J.M. Davidson, eds., (Ann Arbor Science, 1980), p. 189.
18. USEPA Chesapeake Bay Program, "Monitoring Studies of Nonpoint Pollution in Chesapeake Bay Test Watersheds: Final Completion Report," U.S. Environmental Protection Agency, Annapolis, Maryland, (In Press).
19. Bosco, C., G.F. Anderson, and B. Neilson, "Ware River Intensive Watershed Study. 2. Estuarine Receiving Water Quality," final report to Virginia State Water Control Board, (Virginia Institute of Marine Science, Gloucester Point, Virginia), July 1982.
20. Weand, B., and T. Grizzard, "Evaluation of Management Tools in the Occoquan Watershed," final report to Virginia Water Control Board, (Occoquan Watershed Monitoring Laboratory, Virginia Polytechnic Institute and State University, Manassas, Virginia), 1982.
21. Bostater, C., D. McCraney, S. Berlett, and D. Puskar, "Intensive Watershed Study - The Patuxent River Basin," final report to EPA Chesapeake Bay Program, (Maryland Depart of Natural Resources, Annapolis, MD), 1983.

22. Hartigan, J.P., T.F. Quasebarth, and E. Southerland, "Use of Continuous Simulation Model Calibration Techniques to Develop Nonpoint Pollution Loading Factors," Proceedings of Stormwater and Water Quality Management Modeling Users Group Meeting: March 25-26, 1982, EPA 600/9-82-015, U.S. Environmental Protection Agency, Athens, GA., 1982, p. 101.
23. Kirby, R. and E.D. Matthews, "Soil Survey of Anne Arundel County, Maryland," Soil Conservation Service, U.S. Department of Agriculture, 1973.
24. Nielsen, D.R. and J.G. MacDonald, eds., Nitrogen in the Environment, Vol. 1. Nitrogen Behavior in Field Soil, (Academic Press, 1978).
25. Tanji, Kenneth, "Problems in Modeling Nonpoint Sources of Nitrogen in Agricultural Systems," in Environmental Impact of Nonpoint Source Pollution, M.R. Overcash and J.M. Davidson, eds., (Ann Arbor Science, 1980), p. 165.
26. Baker, R.A., ed., Contaminants and Sediments, Vol. 1 and 2, (Ann Arbor Science, 1980).
27. Biggs, R.B., and L.E. Cronin, "Special Characteristics of an Estuary," in Estuaries and Nutrients, B.J. Neilson and L.E. Cronin, eds., (Humana Press, 1981), p. 3.
28. Hegemann, D., A.H. Johnson, and J.D. Keenan, "Determination of Algal-available Phosphorous on Soil and Sediment: A Review and Analysis," J. Environ. Qual., Vol. 12, No. 1, (1983), p. 12.
29. Ott, W.R., Environmental Indices, Theory and Practice, (Ann Arbor Science, 1978).
30. McErlean, A.J., and G. Reed, "Indicators and Indices of Estuarine Enrichment," in Estuaries and Nutrients, B.J. Neilson and L.E. Cronin, eds., (Humana Press, 1981), p. 165.
31. Jaworski, N.A., and O. Villa, Jr., "A Suggested Approach for Developing Water Quality Criteria for Management of Eutrophication," in Estuaries and Nutrients, E.J. Neilson and L.E. Cronin, eds., (Humana Press, 1981), p. 499.
32. Neilson, B.J., and L.E. Cronin, eds., Estuaries and Nutrients, (Humana Press, 1981).

SIMULATION OF THE STORMWATER AND WATER QUALITY ATTRIBUTES OF PONDS WITH HSPF

by

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INTRODUCTION

Stormwater management policies and regulations have been implemented in most Washington Metropolitan Area jurisdictions over the past ten years. The objective of these programs has chiefly been localized flood control and protection against stream bank erosion. The use of Best Management Practices (BMP's) that achieve additional nonpoint pollution control has been encouraged in a few area jurisdictions, but the use of these practices has not been widespread. Most of the 3,100 structures that have been constructed along with new development since the early 1970's are "dry" detention ponds (MWCOC, Aug., 1983). These ponds typically do not incorporate multipurpose design features that would permit them to achieve significant nonpoint pollutant removal.

A major element of the research in urban runoff conducted in Washington under the auspices of EPA's Nationwide Urban Runoff Program (NURP) project involved extensive field investigations of the comparative pollutant removal efficiencies of best management practices (BMP's) that included a variety of stormwater ponds (MWCOC, Dec., 1983). Information on the costs of constructing, operating and maintaining different types of stormwater ponds was also developed. Findings from these investigations can be applied in the planning of individual sites, or in assessing the effectiveness of large scale stormwater management programs for an entire watershed or planning area.

The purpose of this paper is to evaluate the relative pollutant removal performance of wet and dry ponds using data developed from BMP site monitoring and watershed simulation models. The stormwater and pollutant removal capabilities of several types of ponds are presented and contrasted. The capability of widespread employment of stormwater ponds to achieve stream bank erosion and flood control objectives within a 20 mi² watershed is examined. This examination was conducted with a previously calibrated and verified watershed model (MWCOC, Sept., 1983) that represented an application of the HSPF model (Hydrocomp, 1980). It was necessary to explicitly model ponds within HSPF in order to include and contrast the behavior of ponds with different attributes in the analysis. Considerable attention is therefore given in the paper to describing the manner in which ponds were modeled since this may be of interest to the water quality modeling user community.

COMPARATIVE EVALUATION OF STORMWATER PONDS

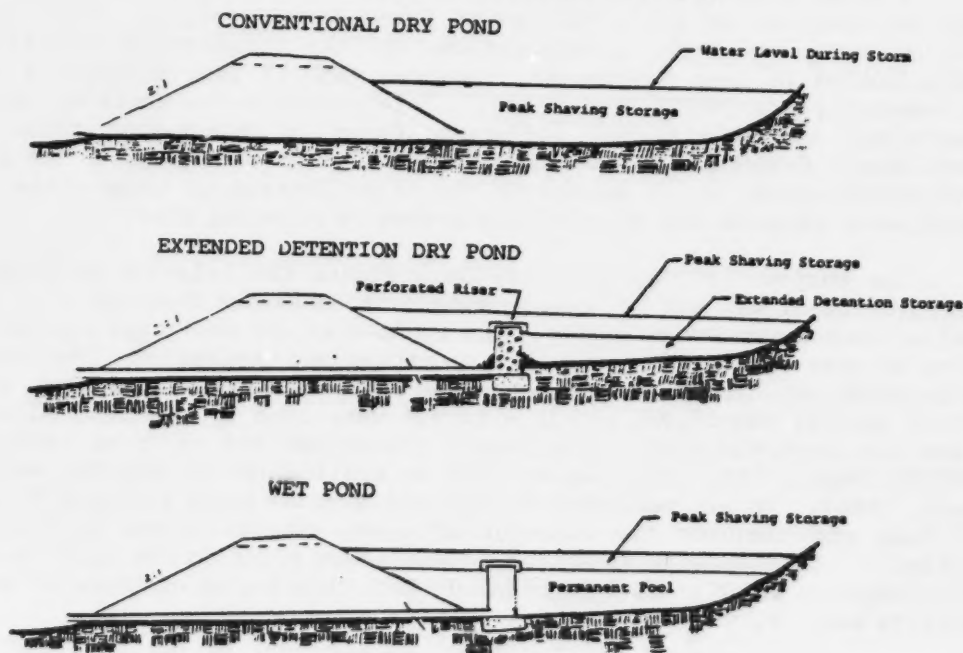
Three types of ponds were evaluated in the NURP field investigations. These were a dry stormwater pond, an extended detention dry pond, and two wet ponds. Schematic diagrams of the principal components of the ponds are presented in Figure 1. All of these ponds were situated in stable suburban neighborhoods and were comparable to structures commonly in use in the area.

Pond Design and Function

"Dry" stormwater detention ponds are designed for the temporary storage of runoff to attenuate or "shave" increases in peak flow associated with land development and increases in impervious area. These structures have been the single most commonly applied stormwater control within the Washington area. Normally empty during dry weather, they fill up rapidly during large storm events. Storage is therefore limited to the volume required to accommodate the design storm (typically 2 years) and release it at pre-development flow rates. The release rate of water is controlled by the size of the outlet pipe or control device that drains the pond. Within the Washington Metropolitan Area, these structures are typically designed to detain stormwater for one to two hours.

An extended detention dry pond is a modified version of a regular dry pond. A major question evaluated under the NURP study was the extent to which the detention time in conventional dry stormwater ponds could be increased to enhance pollutant removal. Based upon previous local investigations (NVPDC, 1980), it was thought that modifications to design that extended detention would provide time for increased settling and removal of pollutants entrained in urban runoff.

FIGURE 1. DIAGRAM OF POND TYPES



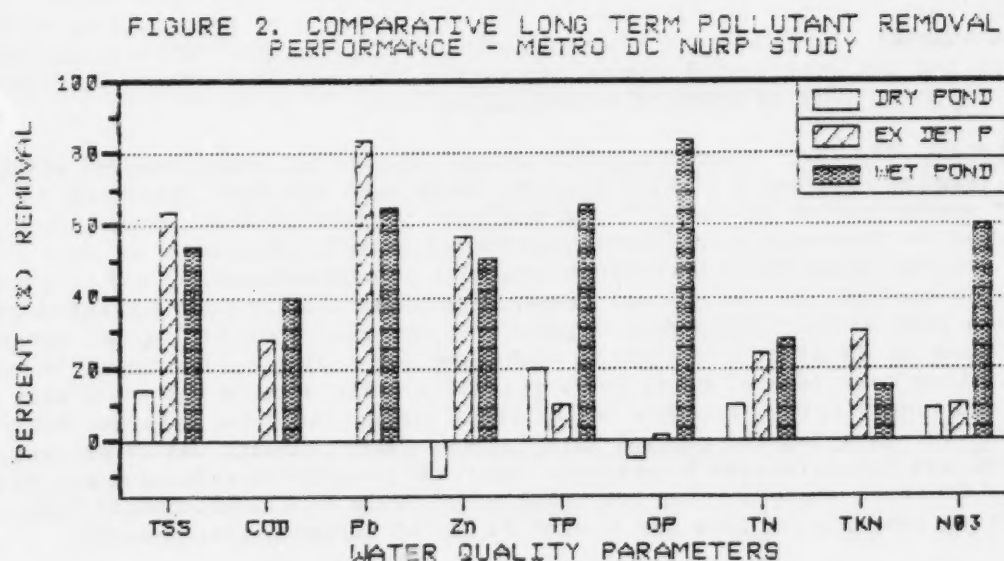
To test this hypothesis, a conventional dry pond was improved to achieve greater detention times. As shown in Figure 1, the outlet is a perforated riser. The extended detention storage is controlled by the flow of water through the perforations.

Stormwater retention or "wet" ponds represent a small but growing segment of local stormwater structures. In addition to providing temporary storage for peak runoff volumes, wet ponds also have additional storage and an outlet device that maintains a permanent pool of water throughout the year. The permanent pool is controlled by the outlet at the top of the unperforated riser. An important hypothesis investigated under the Washington NURP project was that wet ponds had the potential to be very effective in removing pollutants borne in urban runoff. Several different pollutant removal mechanisms were believed to be operative in these ponds including physical settling, chemical flocculation and transformation, and biological uptake. These structures can also serve as a multipurpose landscape feature in some settings as they provide aesthetic, recreational and wildlife attraction benefits to the neighborhood.

Pond Performance

Monitoring stations including flow meters and automated samplers were installed at the inlet and outlet of each pond. This sampling arrangement made it possible to collect flow-composited storm samples. Field sampling was conducted over a 12 to 18-month period, and between 33 to 45 individual storm events were monitored at each site. Long term pollutant removal efficiencies for each pond for total suspended solids, plant nutrients, oxygen demanding materials and trace metals were calculated. This information is summarized in Figure 2.

Dry stormwater pond. Overall, the dry pond did not perform effectively as a best management practice. Removal rates varied considerably during most storms (e.g. sediment removal ranged from positive 98 percent to negative 300 percent).



The general trend, however, was towards negligible or even negative efficiency for most parameters of interest. This is readily apparent in Figure 2 in which the removal rates for the three different types of ponds are compared.

The poor to mediocre performance observed at the dry pond site is probably related to several factors. First, the brief residence time of stormwater within the structure (usually less than 2 hours) severely limits the degree to which physical and chemical pollutant removal can take place. Secondly, the lack of permanent water storage precludes removal by biological means. Thirdly, the substantial number of storms with negative efficiencies suggests that the poorly vegetated pond bed was prone to scour and erosion. Resuspension of previously deposited materials was thought to be an important factor related to poor performance of the pond. The performance of this pond may have been better if the bed had been more stable.

Extended detention dry pond. During the course of the field investigations, stormwater detention times of 6 to 8 hours were achieved. As shown in Figure 2, this extended detention appeared to be promising as a method for improving pollutant removal efficiency. Removal of particulate forms of urban pollutants in the extended detention pond was typically high. For example, sediment removal was approximately 64 percent, organic nitrogen removal was 30 percent, and the reduction in COD was 30 percent. Removal rates for trace metals were also very high. Fifty-seven percent of the zinc and 84 percent of the lead entering the pond was retained. However, removal of soluble nutrient forms was minimal, with long term removal of only 1 percent of the ortho-phosphorus and 10 percent of the nitrate-nitrogen. Removal of total nitrogen (24%) and total phosphorus (10%) was also quite low.

Much of the explanation for the improved pollutant removal performance of these modified dry ponds can be directly attributed to the extended detention achieved. The 6 to 8 hours of detention was sufficient for the effective removal of particulates. Settling column studies conducted by VPI's Occoquan Watershed Monitoring Laboratory (OWML, 1983) supported this conclusion. Other design features that contributed to the high removal of particulates were the paved channel along the pond's bottom between the principal inflow point and the riser, and the established vegetation within the pond bed. These features reduce the likelihood of scour or resuspension.

Wet ponds. The field monitoring data and calculated pollutant removal efficiencies (Figure 2) clearly showed that wet ponds were the most effective for non-point source pollution control. In addition to particulates, these ponds were observed to remove a significant portion of soluble nutrients as well. This high removal rates for soluble nutrients such as ortho-phosphate and nitrate was in sharp contrast to the minimal removals noted at the dry pond and extended detention pond sites. These data suggest the importance of biological uptake in wet ponds as an effective removal mechanism for biologically-available nutrients. Long term removal rates for particulate pollutants in wet ponds were also high and surprisingly comparable to the rates demonstrated for extended detention ponds. For example, the removal rate for suspended sediment was 54 percent, COD removal was approximately 30 percent, and total phosphorus reduction was 64 percent. Observed removal rates for trace metals were also consistently high, with long term removals for zinc and lead at 51 and 65 percent, respectively.

Operative mechanisms for achieving pollutant removal in wet ponds are manifold. It appears that biological uptake is the predominant removal mechanism for nitrate. Deposition, rather than oxidation, seems to be the primary removal mechanism for organic nitrogen. Depositional processes also appear to be fully capable of accounting for the organic phosphorus removal observed. Uptake by algae or rooted aquatic plants seems to be the dominant removal mechanism for ortho-phosphorus. The similarity in removal rates for particulates observed in wet and extended detention ponds implies that depositinal processes, such as settling and flocculation, are more than sufficient to explain the observed patterns of removal for sediments, COD, and trace metals in wet ponds.

In summary, the three types of ponds exhibited markedly different behavior in their ability to remove dissolved and particulate pollutants. Wet ponds displayed the best overall performance. Encouraging results were also obtained for the extended detention dry pond. Although not examined in the field, it was thought that an extended detention wet pond combining the best features of both (i.e., permanent pool and longer drawdown) would further maximize the potential for pollutant removal.

A more complete discussion of the pollutant removal capabilities of these and other urban BMP's can be found in the NURP final report (MWCOC, December, 1983).

CONFIGURATION OF PONDS WITHIN HSPF

Results from the field investigation of ponds were used extensively in the Little Seneca Creek Watershed Management Study (MWCOC, September, 1983; Schueler and Sullivan, 1983). The Little Seneca Creek Study relied heavily on a calibrated and verified HSPF model application. Simulation of the behavior of ponds within HSPF was an important aspect of this work. Processes that occur within open or closed channels or completely mixed lakes are typically simulated in the RCHRES module of HSPF. In this application, pond hydraulics, sediment deposition and water quality processes were directly or indirectly simulated in the RCHRES module. The use of HSPF for the modeling of pond performance proved to be a valuable and innovative way in which to evaluate load reductions and physical impacts upon watershed streams.

Pond Characteristics

Simulation of the dry pond was intended to represent the single purpose stormwater control practices currently required by most local jurisdictions. These ponds provide volume control to the extent that the 2-year peak discharge from a given area is not increased following development. Functionally, they are peak shaving devices that act to prevent added downstream flooding and erosion during storms of two years or greater frequency. Separate ponds were simulated to represent both low and high density development. The principal locational and storage requirements for these dry ponds were set to be consistent with local practices and are shown in Table 1.

Wet ponds were simulated to replicate the types of retention ponds that are becoming more widely used in the Washington area. Separate ponds were again si-

mulated for low and high density development, with the associated description and requirements of each outlined in Table 1. The common denominator for both was a permanent pool with a storage capacity equivalent to one-half inch of runoff over the entire drainage area. Beyond this the low density wet pond had additional storage for runoff in order to maintain the two-year peak discharge rate at predevelopment levels. In the case of high density development, an extended detention wet pond was simulated in order to maximize the water quality benefits of both wet ponds and extended detention. Although extended detention wet ponds were not evaluated in the field investigation, it was assumed that pollutant removal was equivalent to the regular wet ponds, and that the extended detention would be governed by storage volume and the outflow device. The key features of this pond were a larger drainage area, extra storage volume to accommodate extended detention, and a drawdown time of approximately 30 hours.

All of the ponds were considered to be on-site facilities. Stormwater storage volumes and predevelopment discharges for the 2-year storm were obtained from accepted local stormwater engineering and planning guidelines (CH2 Mhill, 1983; DeTullio and Thomas, 1972).

TABLE 1. CHARACTERISTICS OF DRY AND WET PONDS

	DRAINAGE AREA (ACRES)	IMPERVI- OUSNESS (%)	STORAGE VOLUME (CU. FT.)	REMARKS
DRY PONDS				
LOW DENSITY :	25	20%	30,000	NO PERMANENT POOL
HIGH DENSITY:	10	50%	33,000	NO PERMANENT POOL
WET PONDS				
LOW DENSITY :	25	20%	75,000	- PERMANENT POOL = 45,000 CU. FT. & NORMAL DETENTION
HIGH DENSITY:	50	50%	250,000	- PERMANENT POOL = 90,000 CU. FT. & EXTENDED DETENTION TO 30 HRS

Hydrology

Ponds were modeled in the HYDR section of the RCHRES Module, which simulates the hydraulic processes within a free flowing reach or mixed reservoir. The ponds were considered to be completely mixed closed channels with well defined volume/outflow relationships. Pond volume, inflow and outflow are the chief state variables of interest. Flow through the simulated pond is unidirectional. The geometric (depth, surface area, volume) and hydraulic properties of a pond are placed in an HSPF function table (FTABLE) that defines the pond discharge for different depths or volumes of water in storage. An example of an FTABLE describing the geometric and hydraulic properties of a dry pond for low density development is presented in Table 2.

TABLE 3. EXAMPLE OF FTABLE FOR DRY POND

FTABLE				1
ROWS	COLS	***		
13	4			
DEPTH (FT)	SFAREA (ACRES)	VOLUME (AC-FT)	DISCH (CFS)	*****
.000	.000	.000	.000	*****
.1	.23	.023	1.698	
0.5	.23	.115	3.796	
1.0	.23	.230	5.369	
1.5	.23	.345	6.575	
2.0	.23	.460	7.592	
2.5	.23	.575	8.489	
3.0	.23	.690	9.299	
3.1	.23	.713	18.9	
3.2	.23	.736	36.4	
3.5	.23	.781	116.0	
4.0	.23	.920	312.000	
5.0	.23	1.150	864.000	
END FTABLE				1

The outflow relationship contained in the FTABLE was calculated as follows. Pond volume for the 25 acre unit of low density development was determined to be 30,000 ft³. The predevelopment peak flow for the 2-year storm was calculated to be 9.3 cfs. The pond was assumed to be 100 ft long, 100 ft wide, and 3 ft in height. Further calculations to determine outflow rates were based on the general orifice equation (APWA, 1981)

$$Q = CA (2gH)^{0.5} \quad (1)$$

Where: Q = discharge in cubic feet per second
 C = a dimensionless discharge coefficient of 0.75
 A = area of orifice in square feet
 G = gravitational constant of 32.2 feet per second
 H = head of water above orifice in feet

With peak discharge limited to 9.3 cfs and H equal to 3 ft, equation 1 can be rearranged to solve for A, the area of the orifice, as

$$A = \frac{Q}{C (2gH)^{0.5}} \quad (2)$$

Calculation of the orifice diameter is then easily accomplished. Knowing the orifice area A it is possible to calculate the outflow discharge Q for heads of less than 3 ft. For stages greater than 3 ft it was assumed that the pond would overflow, and that one end of the pond would function as a broad crested

weir. Pond overflow is calculated with a general weir equation (APWA, 1981)

$$Q = CLH^{1.5} \quad (3)$$

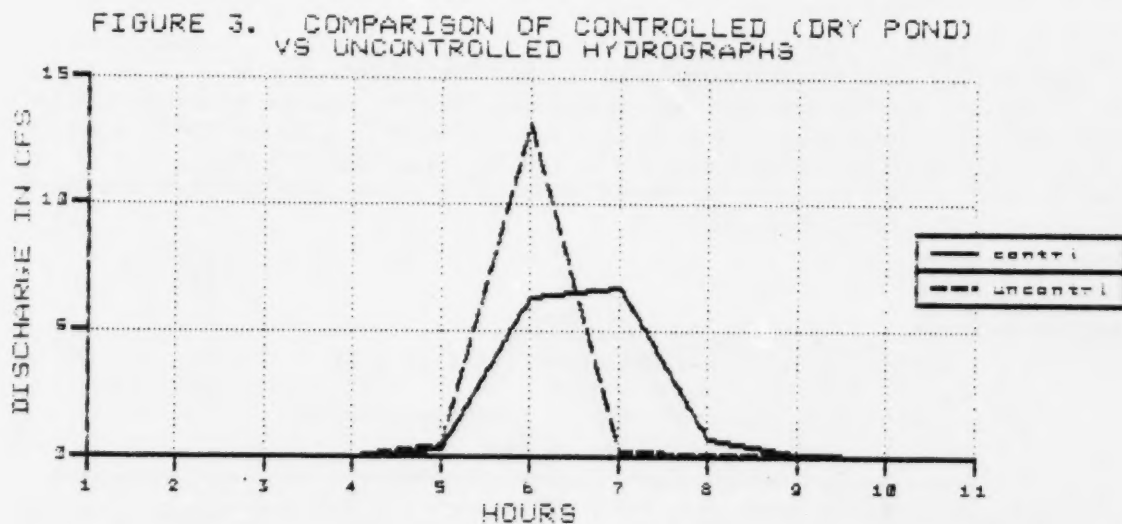
Where: C = a dimensionless discharge coefficient of 3.0

L = length of weir (spillway) in feet

H = head above crest in feet

Discharges calculated in equations 1 and 3 are summed for such overflow situations. The FTABLE is prepared by calculating the appropriate volume and discharge for a full range of depth within the pond and for overflow situations.

Hydrologically, the ponds performed as intended by storing water and releasing it at a controlled rate. The type of stormwater attenuation achieved is given in the hydrographs of Figure 3 which depicts a case of uncontrolled runoff and a case of outflow from the dry pond for the low density development situation. These contrasting hydrographs were produced from a storm of 2.0 inches of rainfall.



Sediment and Water Quality

Average removal efficiencies derived from the Washington NURP work were used to estimate water quality effects of different BMP strategies. In the case of sediment, settling within the ponds was intended to reproduce losses that had been observed in the field. Settling rate studies and other laboratory experiments with clay, silt and sand particles provided additional guidance (OWML, 1983). Based upon this work, different percentages of sand and silt in runoff were made available for settling in the dry and wet ponds. The sediment load in the pond outflow thus reflected a net loss of material that had been deposited in the pond. Removal efficiencies for other water quality constituents were treated similarly. Since the physical, chemical and biological processes af-

fecting these constituents were not modeled per se, removal efficiencies were imbedded in the HSPF NETWORK block as a fixed rate related to pond outflow. The NETWORK block is the section of HSPF input where linkages among land areas, channels, constituents and other processes are described. The multiplication factors typically used in these blocks to account for land area or unit conversions were adjusted to include removal efficiencies as a percent. The removal efficiencies that were used (Table 3) were on the conservative side of efficiencies observed in the field.

TABLE 3. POLLUTANT REMOVAL RATES USED IN MODEL SIMULATIONS

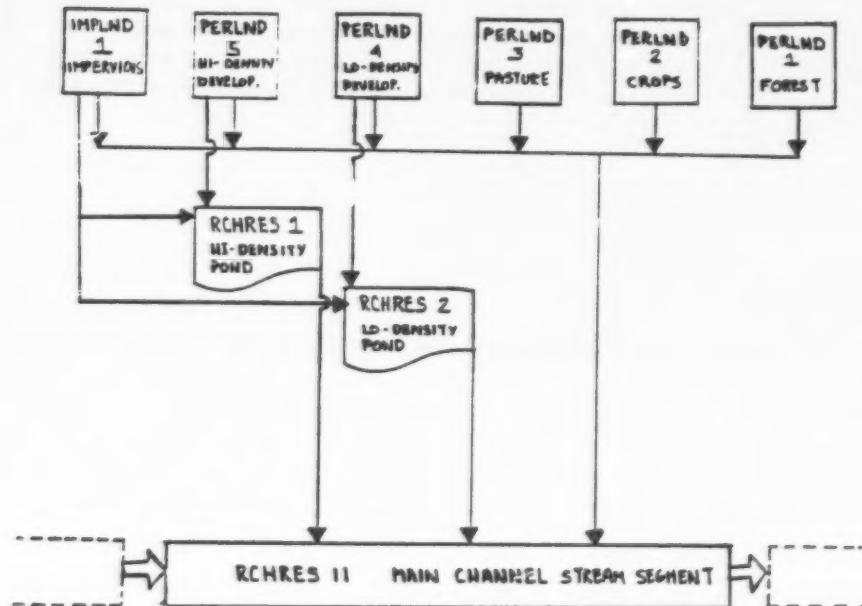
	BOD ---	TP --	TN --	TSS ---
REGULAR DRY POND:	0%	0%	0%	100% SAND
REGULAR WET POND:	50%	50%	30%	100% SAND; 60% SILT
EXTENDED DETENTION WET POND:	75%	50%	50%	100% SAND; 80% SILT

Intrawatershed Links

The individual ponds described in Table 1 and their appropriate drainage, including impervious land, were modeled separately. PERLND (pervious area) and IMPLND (impervious area) inputs of runoff, sediment and constituent loads were routed in the NETWORK block to the appropriate RCHRES segment representing a specific pond. Outflow from these ponds was in turn routed directly to a RCHRES segment representing a tributary or mainstem stream. NETWORK block multiplication factors were used to account for the amount of area within a given subwatershed or drainage area that was governed by the individual pond controls.

The diagram in Figure 4 describes the general way in which the runoff, sediment and water quality constituent products generated within the pervious and impervious land segments were modeled in a subwatershed. Arrows are used to represent connections between HSPF modules where multiplication factors are utilized. The extensive use of multiplication factors in determining distributions within this configuration is very important.

FIGURE 4. DIAGRAM OF CONNECTIONS WITHIN HSPF

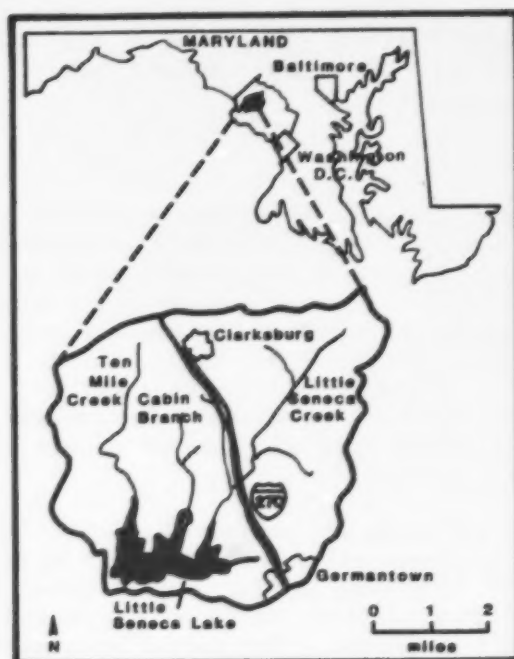


APPLICATIONS OF HSPF TO EVALUATE IMPACTS PROJECTED FOR MASTER PLAN DEVELOPMENT OPTIONS

The Little Seneca Creek watershed presented an opportunity to use the results from the field investigations of ponds and the calibrated watershed model (HSPF) in the development of a Master Plan. The Little Seneca Creek watershed (Figure 5) lies approximately 25 miles to the northwest of Washington, D.C. in an area of suburban Montgomery County, MD that is experiencing rapid development. The 20 mi² area drains to a 600 acre water supply and recreational lake that is currently under construction. Because of the lake, an interstate highway that bisects the watershed, and associated pressure for development, a revised Master Plan is being formulated for the area. Several land use planning alternatives ranging from agricultural preservation to a rather dense level of commercial and residential development are under consideration. Concern about the environmental consequences of these alternatives upon watershed tributaries and the proposed lake prompted the evaluation of alternatives and remedial stormwater management practices.

The use of different types of ponds for the control of stormwater runoff in new development was simulated in order to contrast control effectiveness on a basin-wide level. Model calculations were made for a full year that represented average climatological conditions. The projected effectiveness of implementing dry ponds vs. wet ponds for the "ultimate" land use alternative, the alternative under consideration with the densest development, is presented in Table 4. The

FIGURE 5. LITTLE SENECA CREEK WATERSHED



dry pond scenario was intended to represent the effects of stormwater management as required by existing regulations and practices. Consequently, dry ponds were earmarked to control 70 percent of new development. This approximation reflects the level of new development actually receiving controls under the existing control regulations which frequently permit waivers of stormwater requirements under certain conditions. In contrast, 100 percent coverage of new development was assumed in the wet pond scenario. The uncontrolled or "no pond" situation for the ultimate development land use alternative, a projection representing the

TABLE 4. SUMMARY OF PROJECTIONS FOR CONTROL OPTIONS

	IMPACTS TO TRIBUTARIES				IMPACTS TO THE LAKE				
	PEAK Q (CFS)	LOW Q (CFS)	STRMBNK EROSION (T/MI/YR)	CHANNEL DEPOSIT (T/MI/YR)	TP LOAD (1000'S LBS/YR)	TN LOAD (1000'S LBS/YR)	BOD LOAD (T/YR)	SED LOAD (T/YR)	AVG CHL _a (UG/L)
PREDEVELOPMENT: (100% FOREST)	1295	5.8	80	13	3.0	74	336	4.5	6
EXISTING: (1980 LAND USE)	1400	5.5	115	122	14.5	221	810	20.5	26
ULTIMATE: (NO CONTROLS)	1947	4.4	248	83	14.6	243	1083	16.5	26
ULTIMATE: (DRY PONDS)	1632	4.5	241	41	14.6	242	1083	14.6	26
ULTIMATE: (WET PONDS)	1348	4.9	188	16	7.0	168	526	10.5	12

existing land use, and one representing a predevelopment (100% forest) condition are provided in Table 4 as an additional basis for comparison. Individual measures of projected impact upon tributary streams and upon the proposed Little Seneca Creek Lake were identified and calculated so that stormwater management options could be evaluated in a meaningful manner. Impacts to tributaries and the lake are discussed separately below.

Impacts to Tributaries

The effect of land use change on peak storm discharge and potential flooding is one of the major concerns of a stormwater management program as it can have a direct impact upon human safety and property damage. It has been demonstrated that the implementation of detention basins, especially in downstream subareas, can have the unintended effect of increasing peak flows at downstream points (Travers, R.G. and Chadderton, R.A., 1983). An evaluation of whether or not the ponds were actually shaving peak flow rates was very important. Several physical processes such as low flow, streambank erosion and sedimentation also play an important role in determining the quality of the aquatic and riparian environment along streams.

As shown, the use of wet ponds was projected to keep the peak discharge below the existing or predevelopment rate. Flood control objectives were thus realized. In contrast, the dry pond option showed some reduction from the uncontrolled postdevelopment situation, but a net increase of 17% over the predevelopment rate was still projected. The better results associated with the wet pond strategy are in large part due to the simulation of extended detention wet ponds for high density development. The extended drawdown of these ponds provides a great deal of attenuation that has a very positive effect in dampening downstream peak flow rates. The possibility of stormwater ponds providing a net contribution to downstream flooding is greatly reduced with the use of extended detention wet ponds. The inability of the dry pond strategy to maintain predevelopment peak flow levels was in part due to the 70 percent coverage assumption that was mentioned above. Beyond this the one to two hour detention periods are often not long enough to have much of a positive effect on downstream flooding.

The effect of the ponds on low discharge during the driest month was not pronounced, but slightly more favorable with wet ponds. Wet ponds would play a role, albeit a minor one in the case simulated, in contributing to higher levels of baseflow during dry periods.

The wet ponds do provide a large measure of protection against streambank erosion. However, an increase in streambank erosion is still projected for this level of urbanization notwithstanding the widespread application of these ponds. The projected increase in streambank erosion occurs because the total amount of stormwater generated from the new development is increased. This volume of water is sufficient, even as it is attenuated in the ponds, to generate extensive streambank erosion because of the increased frequency and longer duration of high velocity "scouring" flow. The deposition of sediments in stream channels was projected to be sharply reduced under both of the pond options. Wet ponds would appear to be capable of bringing sedimentation rates down nearly to the predevelopment level. The large amount of agricultural land with high

sediment yield provides some explanation for the high sedimentation rate associated with the existing land use situation. Since the tributary streams are swift flowing and well aerated, there were no discernable changes in dissolved oxygen concentrations.

Impacts to Little Seneca Creek Lake

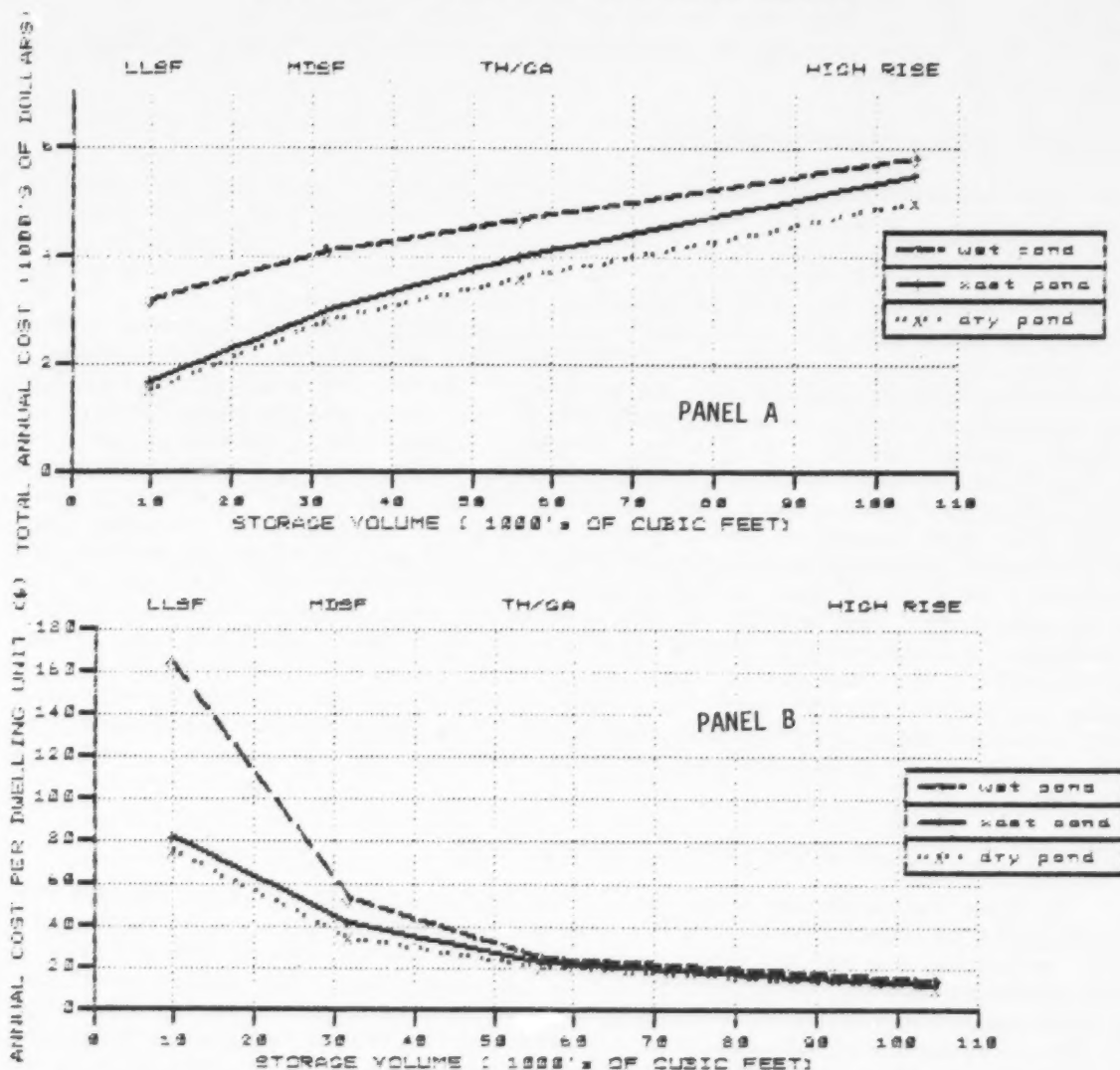
Water quality in the proposed Little Seneca Creek Lake was an important consideration in the master plan process. Several measures that were thought to either directly or indirectly affect the lake and lake water quality were identified and included in the evaluation of control options. The total annual loads of sediment, BOD, nitrogen and phosphorus are shown in Table 4 to be greatly reduced with the wet ponds. Projected reductions associated with the dry ponds, however, were negligible. Taking these projections one step further, several preliminary conclusions can be drawn. The wet ponds would act to limit the delivery of oxygen demanding material to the lake, thereby benefiting dissolved oxygen levels. Similarly, sediment loads would be less and the consequences of this would be better water clarity (less turbidity) and a reduced amount of sedimentation on the lake bottom. Projected total phosphorus loads were used to estimate mean annual chlorophyll-a levels for the lake. The estimation of chlorophyll-a served as a substitute measure for algae production. Estimates were based upon applying the biologically available fraction of the total phosphorus load and the relevant geometric and hydraulic data to the Vol-lenweider Eutrophication Model (USEPA, 1977). The resulting levels of chlorophyll-a were associated with trophic conditions as shown in Table 4. Besides the predevelopment condition, the wet pond option was the only projection that provided for a non-eutrophic condition. This reflected the higher phosphorus removal rates simulated for wet pond structures vis a vis dry or extended dry ponds.

Comparative Costs of Dry and Wet Stormwater Ponds

A major focus of the Washington area NURP study was directed towards the development of a reliable method for estimating the costs involved in constructing, operating and maintaining stormwater ponds (MWCOCG, June, 1983). From a management viewpoint, this information is important since the full cost of implementing and operating a stormwater control is usually the controlling factor in the development of an effective and feasible stormwater management strategy. The full cost of a stormwater pond was defined as the initial capital cost of construction plus the subsequent O&M payments. Estimates assumed a twenty-year project life, and 8 percent discount rate, and a 1980 price base.

The comparative costs of stormwater ponds in the Washington metropolitan area are illustrated in Figure 6. In panel A, the total annual payment for individual dry, wet and extended detention ponds are shown in relation to land use/storage requirements. Panel B displays the same information, but in terms of annual payment per dwelling unit. Land use densities included on this figure include large lot single family (LLSF), medium density single family (MDSF), townhouse/garden apartment (THGA), and high rise/high density (HI RISE). Differences in pond costs are largely attributable to the extra storage requirements of the wet and extended detention wet ponds. As can be seen, a wet pond of similar size costs from 26 to 46 percent more than single-purpose dry

FIGURE 6. COMPARISON OF POND COSTS



ponds. Similarly, extended detention ponds typically entail 10 to 12 percent more cost than single purpose dry ponds. In general, the cost differential between wet ponds or extended detention ponds and the single-purpose dry pond narrowed as land use density increased. Figure 6 also suggests that economies of scale can be achieved in stormwater pond construction. As an example, larger, centralized off-site wet ponds were found to be as economical as smaller, de-centralized dry ponds in the Little Seneca Lake Study (Schueler and Sullivan, 1983). Finally, the greater cost of wet and extended detention wet ponds may be offset to some degree by the realization of aesthetic amenities including recreation, landscape, and wildlife benefits attributed to these multi-purpose ponds.

CONCLUSION

Field investigations revealed a marked difference in the pollutant removal abilities of dry ponds and wet ponds. Removal efficiencies were enhanced in ponds where settling and biological processes were active. The simulation of pond behavior in HSPF proved to be a valuable addition to watershed management planning. The cumulative effect of a widespread application of BMP's was systematically evaluated within individual tributaries and at the downstream terminus of the watershed. The simulation of dry and wet ponds allowed relative differences in peak discharge rates, streambank erosion, pollutant transport, and potential levels of algal production to be calculated. The superior performance of wet ponds over dry ponds in the basin-wide assessment was demonstrated for nearly all of the measures of interest. The cost of implementing a wet pond management strategy was shown to be greater than costs associated with implementing the more conventional dry ponds. However, cost differences are minimized with increasing density, and there also appear to be other benefits associated with wet ponds.

This approach to watershed modeling and the simultaneous evaluation of control options and land use planning alternatives was very well suited for the environmental and planning issues addressed in the Little Seneca Creek Watershed Study. Several other questions or issues that could potentially be addressed with HSPF include: 1) the placement and sizing of ponds to minimize peak discharge rates at all places within a watershed; 2) the explicit modeling of sediment and water quality processes within ponds; and 3) the evaluation of design features to enhance pond performance.

REFERENCES

American Public Works Association. "Urban Stormwater Management: Special Report No. 49", Chicago, IL, 1981.

CH2MHILL. "Seneca Phase II Watershed Study", report prepared for the Maryland-National Capital Park and Planning Commission, Silver Spring, MD, 1983.

DeTullio, D. and Thomas, R. "Stormwater Management Cost Study", USDA Soil Conservation Service, report prepared for the Maryland-National Capital Park and Planning Commission, Silver Spring, MD, 1977.

Hydrocomp, Inc., "User Manual for Hydrologic Simulation Program - Fortran (HSPF)", U.S. Environmental Protection Agency, Environmental Research Laboratory, Athens, GA, 1980.

Metropolitan Washington Council of Governments, "An Evaluation of the Costs of Stormwater Management Pond Construction and Maintenance", Washington, D.C., June, 1983.

Metropolitan Washington Council of Governments, "Potomac River Water Quality - 1982: Conditions and Trends in Metropolitan Washington", Washington, D.C., August 1983.

Metropolitan Washington Council of Governments, "The Seneca Creek Watershed Management Study: Final Report", Washington, D.C., September 1983.

Metropolitan Washington Council of Governments, "Urban Runoff in the Washington Metropolitan Area", Final NURP Report, Washington, D.C., December 1983.

Occoquan Watershed Monitoring Laboratory (OWML), "Final Contract Report on Metropolitan Washington Urban Runoff Demonstration Project", report prepared for Metropolitan Washington Council of Governments by VPI & SU, OWML, 1983.

Schueler, T.R. and Sullivan, M.P., "Management of Stormwater and Water Quality in an Urbanizing Watershed", Proceedings of 1983 International Symposium on Urban Hydrology, Hydraulics and Sediment Control, University of Kentucky, Lexington, KY, 1983.

U.S. EPA, "Water Quality Assessment: A screening Method for Nondesignated 208 Areas", EPA-600/9-77-623, Environmental Research Laboratory, Athens, GA, 1977.

Travers, R.G. and Chadderton, R.A., "The Downstream Effects of Storm Water Detention Basins", Proceedings of International Symposium on Urban Hydrology, Hydraulics and Sediment Control, University of Kentucky, Lexington, KY, 1983.

KINEMATIC ANALYSIS OF DETENTION STORAGE

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ABSTRACT

The role of detention storage in attenuating peak floods is investigated in a general manner using the kinematic flow equations. A dimensionless form of the equations is used to provide guides to the design of channels and ponds in order to optimize surface or channel detention or pond sizes. Resulting hydrograph shapes for different types of control are presented.

The paper looks at channel storage as well as pond storage. Channel storage and catchment (on site) storage are achieved by retarding the runoff such as by lengthening the flow path, reducing slope or increasing roughness. These effects are achieved with numerical models based on the kinematic equations.

A kinematic model for simulating conduit flow between detention storage ponds is presented. The model accounts for fluctuating storage volumes and inflows and outflows.

INTRODUCTION

Many research projects have been based on the use of mathematical models to study the runoff off urban and rural catchments. In fact the number of such reports could approach the number of catchments or candidates, whichever is lesser, multiplied by the number of models available. Much of the research is published but even more is in a form of little value to the practicing engineer. Sensitivity studies and model calibration have to be performed from scratch for new drainage designs.

It has been the objective of the Water Systems Research Programme at the University of Witwatersrand to condense as much of the information from catchment models as possible into simple relationships for use by practicing engineers (Stephenson, 1982). Such design aids have included peak flow rates and identification of critical storm durations for any catchment. Hydrographs for simple and complex catchments have been generated (Constantinides and Stephenson, 1982) and the effects of storm dynamics and movement have been identified. All these studies have been performed in a generalized way and the results plotted in the form of dimensionless charts.

The kinematic equations have been used in analytical and numerical form for many of the studies. The following study attempts to generalize the effects of channel and pond detention storage on runoff rate and evaluates the relative effects on storage volume and channel capacity.

STORAGE FUNCTIONS

Starting with the St. Venant equations it is easy to see the components of storage:

$$\frac{\partial A}{\partial t} = - \frac{\partial Q}{\partial x} \quad (1)$$

$$\frac{\partial v}{\partial t} = g(S_o - S_f) - g \frac{\partial y}{\partial x} - v \frac{\partial v}{\partial x} \quad (2)$$

The first equation is the continuity equation and the second the so-called dynamic equation. In fact the first equation is not a storage equation, it represents the rate of change in cross sectional area of flow as a function of inflow and outflow. The second equation contains more about the distribution of storage. The last two terms represent the wedge component of storage, which are absent in the kinematic equations. The kinematic equations therefore treat storage as a prism, with storage in blocks and no allowance for difference in slope between bed and water surface is made.

Since the second equation is replaced by a friction equation and $S_o = S_f$ in the kinematic equations, only the first equation can be used to calculate storage changes.

Comparison with Muskingum Equations

The continuity equation may be written as

$$\frac{O - I}{\Delta x} + \frac{A_2 - A_1}{\Delta t} = 0 \quad (3)$$

where O is outflow. I is inflow over a reach of length Δx , and A_1 and A_2 are the cross sectional areas before and after Δt respectively. If $O = (O_1 + O_2)/2$ and $I = (I_1 + I_2)/2$ and $A\Delta x$ is replaced by S , the storage which is a function of A_1 and A_2 , which in turn are functions of flowrate, eg. $S = xI + (1-x)O$, then the above equation becomes the one frequently used for open channel routing,

$$O_2 = c_1 I_1 + c_2 I_2 + c_3 O_1 \quad (4)$$

where c_1 , c_2 and c_3 are functions of x . The latter equation is referred to as Muskingum's equation. If $x = 0$ the routing equation corresponds to level pool or reservoir routing. The more general equation with $x = 1/2$ represents a 4-point numerical solution of the continuity equation as employed in kinematic models (Brakensiek, 1967).

CHANNEL RESISTANCE AND STORAGE

Channel storage performs a similar function to pond storage, and there are many analogies to be drawn between the two. Channel storage is a function of friction resistance and channel shape.

The form of friction equation, as well as the friction factor, affect the reaction speed of a catchment and the volume stored on the catchment. The excess rain stored on the catchment, whether in channels or on planes, is a form of detention storage, and as such, affects the concentration time and consequently the peak rate of runoff.

Both the exponent of y in the equation $q = zy^m$ and the roughness coefficient z affect the flow depth - discharge relationship. Some friction formulae used in stormwater drainage practice are listed below (SI units)

Darcy	$Q = (8/f)^{1/2} A(RSg)^{1/2}$	(5)
-------	--------------------------------	-----

Chezy	$Q = 0.55 CA(RS)^{1/2}$	(6)
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$$\text{Manning} \quad Q = A R^{2/3} S^{1/2} / n \quad (7)$$

$$\text{Strickler} \quad Q = 7.7 A (R/k)^{1/6} (RSg)^{1/2} \quad (8)$$

R is the hydraulic radius A/P where A is the area of flow and P the wetted perimeter. S is the energy gradient, f is the friction factor and k is a measure of roughness.

If it is assumed that the friction factors are constant then Q is proportional to $S^{1/2}$ in all cases. This is true for high Reynolds numbers but for shallow flow the friction factors should be increased.

By comparing the Manning and Strickler equations it may be deduced that $n = 0.13 k^{1/6} / g^{1/2}$ and by comparing with the Darcy equation, $n = (f/8g)^{1/2} R^{1/6}$. The latter will be found to hold reasonably well for high Reynolds numbers but for lower Reynolds numbers f is known to increase and n should increase too. The following table indicates increasing values of n for a wide channel with an initial n of 0.02 at 1 m depth, a slope of 1/100 and decreasing depths. The n is calculated from f which in turn is calculated from the Colebrook-White equation.

Water depth, m	Reynolds No.	Manning n	Darcy f
1.0	1 000 000	0.02	0.03
0.1	40 000	0.023	0.09
0.01	1 000	0.040	0.6
0.001	20	0.062	3.0

The Manning roughness coefficient does not vary as much as the Darcy coefficient with reducing Reynolds number. On the other hand, the square root of Darcy's coefficient is employed to calculate flow rates which compensates to a small degree, but generally it would appear the Manning coefficient is a more reliable number to use for channels (at high Reynolds numbers) and overland flow (at low Reynolds numbers), as a more reliable estimate of n is possible than f, since n does not vary as much.

The variation in peak flow from catchments depends on the roughness coefficient and the exponent m of R or y . This is largely due to the attenuating effect of friction resulting in a larger time to equilibrium. The rain excess intensity-duration relationship is therefore required to evaluate the effect of each coefficient on peak runoff rate and maximum catchment storage. The following expression for excess rainfall intensity is assumed:

$$i_e = \frac{a}{(b + t_d)^c} \quad (9)$$

In this equation it is customary to express i_e and a in mm/h and b and t_d in hours. t_d is the storm duration, assumed equal to time of concentration t_c for maximum peak runoff off a simple catchment. Starting with the kinematic equation for continuity

$$\frac{\partial y}{\partial t} + \frac{\partial q}{\partial x} = i_e \quad (10)$$

and a general resistance equation

$$q = zy^m \quad (\text{m-s units}) \quad (11)$$

then it may be shown that $t_c = (L/z i_e^{m-1})^{1/m}$ (seconds) where q is the runoff rate per unit width of catchment and y is flow depth.

The rising limb of the hydrograph is given by the equation

$$q = z(i_e t)^m \quad (\text{m}^3/\text{s}/\text{m}) \quad (12)$$

and another expression may be derived for the falling limb. In Fig. 1 are plotted dimensionless hydrographs to illustrate the effect of m on the shape of the hydrograph. The graphs are rendered dimensionless by plotting $Q = q/i_e L$ against $T = t/t_c$. m is used as a parameter. Thus $m = 1/2$ represents closed conduit or orifice flow, $m = 1$ represents a deep vertical sided channel, $m = 3/2$ represents a wide rectangular channel according to Darcy, or a rectangular weir, $m = 5/3$ represents a wide rectangular channel according to Manning and $m = 5/2$ represents a triangular weir.

The graphs immediately indicate the effect of m on catchment detention storage. The smaller m , the greater the storage. Thus provided storage is economical, throttled outflows increase storage and reduce discharge rate (which is not immediately apparent from these graphs as they are plotted

relative to excess rainfall intensity). In practice the concentration time increases the greater the storage so that lower intensity storms became the design storms. This has a compound effect in reducing flow rates since losses increase and it is possible that the entire catchment will not contribute at the peak flow time.

A general solution for peak flow and storage in terms of rainfall intensity-duration relationships is derived below. Solving (9) with $t_d = t_c$ for maximum rate of runoff per unit area, generalized by dividing by a ,

$$\begin{aligned} q_m/aL = i_e/a &= \frac{1}{\left(b + \frac{(L/zi_e^{m-1})^{1/m}}{3600}\right)^c} \\ &= \frac{1}{\left(b + \frac{[L/z(a/3600000)^{m-1}]^{1/m}}{3600(i_e/a)^{1-1/m}}\right)^c} \end{aligned} \quad (13)$$

The term L/za^{m-1} is referred to as the lag factor. The constants are introduced since a is in mm/h, and time of concentration is in seconds while (9) requires hour units. The maximum peak flow factor i_e/a is plotted against lag factor in Fig. 2, since it is not easy to solve (13) directly for i_e/a .

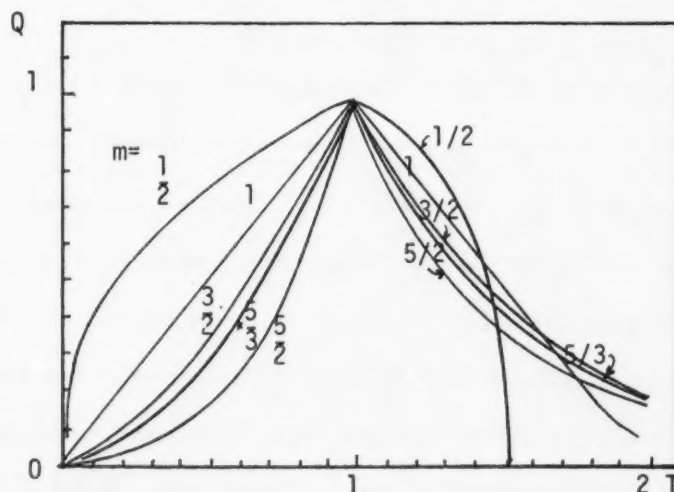


Fig. 1 Hydrograph shapes for different values of m in $q = zy^m$

i_e/a and s/a

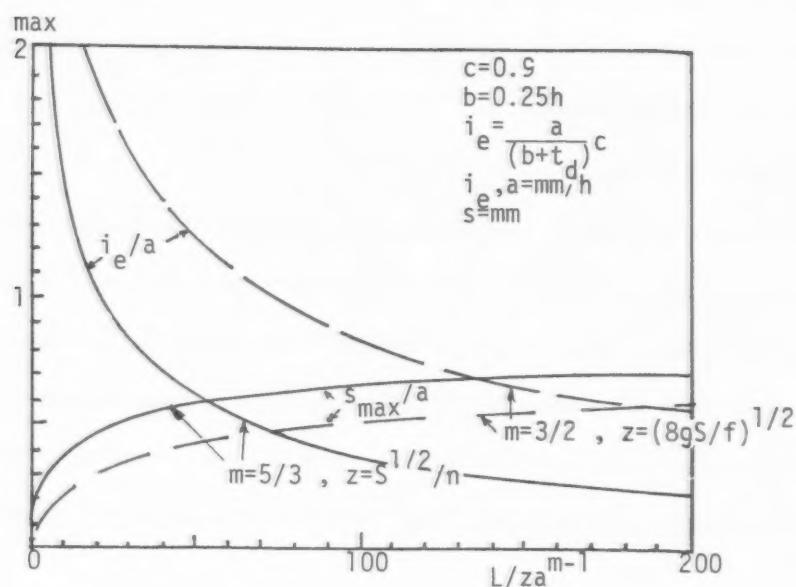


Fig. 2 Peak flow and storage versus lag factor

An expression for the corresponding catchment storage is derived below. At equilibrium the flow per unit width at a distance x down the catchment is

$$q = i_e x$$

$$= z y^m$$

therefore $y = (i_e x/z)^{1/m}$.

Integrating y with respect to x yields the total volume down the catchment

$$V = \frac{Lm}{m+1} (i_e L/z)^{1/m}$$

or in terms of the average depth of storage $s = V/L$,

$$s/a = \frac{m}{m+1} \left(\frac{i_e}{a} \right)^{1/m} \left(\frac{L}{z(a/3600000)^{m-1}} \right)^{1/m} \frac{1}{3600} \quad (14)$$

where s is in mm, and i_e and a are in mm/h. s/a is also plotted against lag factor in Fig. 2. It will be observed that average storage depth does not increase in proportion to L/za^{m-1} , in fact the rate of increase reduces beyond $L/za^{m-1} = 50-100$, and the rate of reduction in peak flow i_e/a also decreases

beyond this figure indicating reducing advantage in increasing channel length or roughness ($z = S^{1/2}/n$). Since total channel cost is a direct function of storage capacity it would appear to be an optimum at some intermediate value of L/za^{m-1} if there is a cost associated with peak discharge e.g. culverts or flooding downstream.

Note that infiltration after the rainfall stops, is neglected in the above analysis. Inclusion of that effect would lower the i_e/a and s/a lines to the right, implying a larger L/za^{m-1} is best.

The model provides an indication of total storage in the system. The location (and volume) of storage could be further optimized using dynamic programming methods or by detailed modelling. It should be found generally that it is most economical to provide pond storage ($m = 1/2$) at the outlet, whereas channel or catchment storage ($m = 5/3$) is most economic at the head of the system.

KINEMATIC EQUATIONS APPLIED TO CLOSED CONDUITS

INTERCONNECTING RESERVOIRS

If the open channel kinematic equations are applied to closed conduit flow the problem becomes a steady state flow one since flow rates become independent of cross section. This is provided the conduits remain full and there are no storage ponds at nodes joining conduits. If one permits storage variation at nodes one has the reservoir-pipe situation encountered in water supply which is often analyzed employing pseudo-steady flow equations.

The continuity equation becomes (see Fig. 3)

$$(Q_{i+1} - Q_i) - q_i + A_i \frac{dh_i}{dt} = 0 \quad (15)$$

where the reservoir surface area A_i replaces $B dx$ in the open channel continuity equation where B is the catchment width. q is the reservoir inflow here. The dynamic equation is replaced by

$$Q_i = z A^m \quad (16a)$$

where A is the (constant) conduit cross sectional area. Since the kinematic equations omit the dependency of Q on head difference Δh , the latter equation assumes the head gradient along the pipe

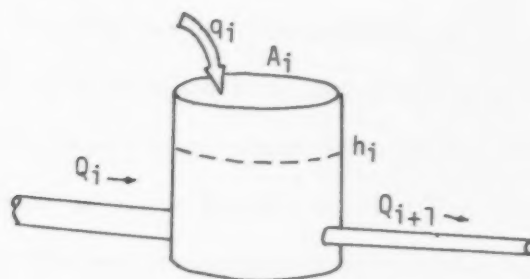


Fig. 3 Input-output node storage

equals the pipe gradient, i.e. free-surface just full flow. Since A is a constant it is relatively easy to replace the last equation by one of the form

$$Q_i = zA h_i^m \quad (16b)$$

This equation is applicable to free discharge from an orifice or over a weir. One more applicable to conduit flow would be

$$Q = zA (h_{i-1} - h_i)^m \quad (16c)$$

Any one of the above three equations could be applicable in stormwater drainage. For channel or overland flow (16a) applies, for complete storage control (16b) applies and for closed conduit control (16c) is applicable. The latter form of equation has in fact been employed in water reticulation pipe network analysis. It can be applied in storm drainage to closed systems (not of great interest in stormwater management practice) or to pipe-reservoir problems. Surface detention and artificial detention storage ponds can be handled in an overall flow balance employing the closed conduit kinematic method. It should be noted that the numerical instability problems associated with solution of the open channel kinematic equations are absent. Time steps can be much larger than for open channel kinematic modelling.

Storage fluctuations may be computed in steps and the affect of changes in pond water levels on flows in conduits can be accounted for.

One possible application of such a program is to an interconnected pond system with reversible flows in conduits. Overload from one pond can be forced back to another pond. Such situations can readily arise from spatially variable storms and possible from travelling storms.

Off-channel storage can also be accounted for. Such ponds have the advantage that water level variations are not as marked as the head variations in the drain pipes (which may in fact be surcharged). This is due to the reversible head loss between the main conduit and the pond.

The simplified layout in Fig. 4 was analyzed employing the kinematic closed conduit continuous simulation program 'KINC'. Input and output are appended to illustrate the simplicity in this type of analysis. Flow reversal, pond level variations and the large attenuation in peak flow will be observed due to the ponds (from $5.6 \text{ m}^3/\text{s}$ down to $1.5 \text{ m}^3/\text{s}$). By adjusting individual pond areas and conduit sizes an optimum design could be achieved for any design storm input. A sensitivity analysis for alternative storms (e.g. larger deviation or spatially variable) would then be performed.

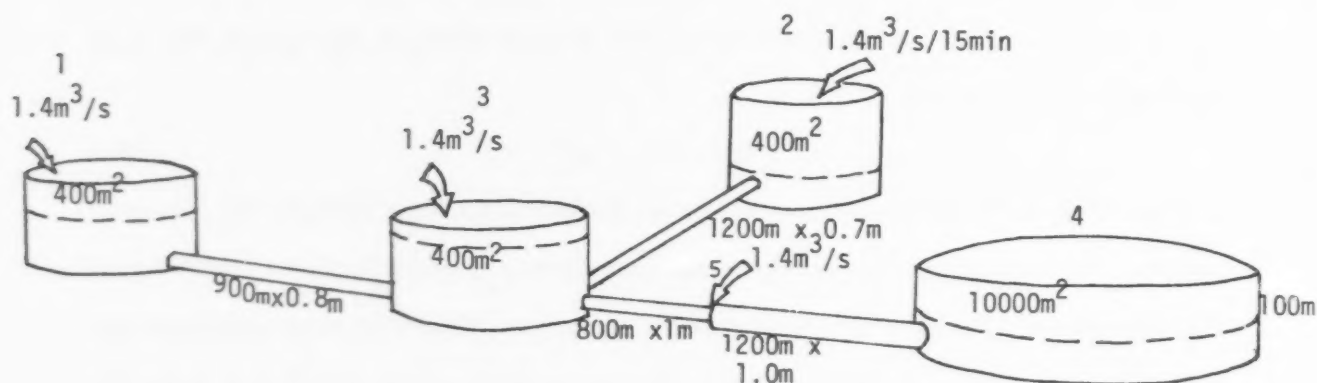


Fig. 4 Conduit and storage storm drain network.

REFERENCES

- Brakensiek, D.L., 1967. Kinematic flood routing. Trans. Am. Soc. Agric. Engrs., 10 (3), p. 340-343.
- Constantinides, C.A. and Stephenson, D., 1982. Dimensionless Hydrographs using Kinematic Theory. Water Systems Research Programme, University of the Witwatersrand, Johannesburg, Report 5/1982.
- Stephenson, D., 1982. Peak Flows From Small Catchments Using Kinematic Theory. Water Systems Research Programme, University of Witwatersrand, Johannesburg, Report 4/1984.

KINC program

```

10 ! KINC KINEMATIC CONTIN SIMU
LN OF NETWORKS WITH STORAGE
20 DIM C(50),Q(50),H(50),Q2(50)
,S2(50),S4(50),F(50)
25 DIM J1(50),J2(50),D(50),X(50)
,A(50),M1(50),M2(50,9)
30 DISP "NAME OF NETWORK";
40 INPUT N$
41 DISP "DRAWOFF DURN,min,SIM D
URN,min,DT,min,DRAWOFF m3/s"
;
42 INPUT T3,T4,T5,Q2(1)
45 T3=T3*60
46 T4=T4*60
47 T5=T5*60
50 DISP "NPIPES,NODES,NRES,DARC
Yf";
60 INPUT P,J,J3,F1
62 DISP "INITL WATER LEVELm,SUR
FACE AREA m2";
64 FOR L=1 TO J3
65 DISP L;
67 INPUT H(L),A(L)
68 NEXT L
69 G=9.8
70 S=3.14159*2*G/8
75 DISP "TOPN,BOTN,Lm,Dm,OUTFLO
Bm3s";
80 FOR K=1 TO P ! PIPE DATA
85 DISP K;
90 INPUT J1(K),J2(K),X(K),D(K),
Q1
100 Q2(J2(K))=Q1
110 F(K)=F1
120 Q(K)=3.14159*D(K)^2/4
125 C(K)=S*D(K)^5/F(K)/X(K)
127 IF J2(K)<J3 THEN 140
130 H(J2(K))=H(J1(K))-1/C(K)*Q(K)
^2
140 NEXT K
142 FOR L=1 TO J
143 M1(L)=0
144 FOR M=1 TO P
145 IF J1(M)=L THEN 147
146 IF J2(M)<>L THEN 149
147 M1(L)=M1(L)+1
148 M2(L,M1(L))=M
149 NEXT M
150 NEXT L
158 W=1.3
160 T0=.0001
165 T1=.01
170 N0=SQR(J)+5
180 N1=SQR(J)+10
190 N2=0
192 N3=0
200 PRINT "PIPENET",N$
202 FOR T6=T5 TO T4 STEP T5
204 IF T6<=T3 THEN 210
206 FOR L=1 TO J
208 Q2(L)=0
209 NEXT L
210 FOR I=1 TO N0
220 N2=N2+1
230 FOR K=1 TO J ! NEW H BY SOR
240 S2(K)=0
250 NEXT K
260 FOR K=1 TO P
270 S2(J1(K))=S2(J1(K))-C(K)/ABS
(Q(K))
280 S2(J2(K))=S2(J2(K))-C(K)/ABS
(Q(K))
290 NEXT K
300 FOR K=1 TO N1
310 C2=0
320 S3=0
330 N3=N3+1
335 IF J3+1>J THEN 480
340 FOR L=J3+1 TO J
350 S4(L)=0
360 FOR M3=1 TO M1(L)
365 M=M2(L,M3)
370 IF J1(M)<>L THEN 390
380 S4(J1(M))=S4(J1(M))+C(M)/ABS
(Q(M))*H(J2(M))
390 IF J2(M)<>L THEN 410
400 S4(J2(M))=S4(J2(M))+C(M)/ABS
(Q(M))*H(J1(M))
410 NEXT M3
420 D2=H(L)
430 H(L)=H(L)*(1-W)+W*(Q2(L)-S4(
L))/S2(L)
440 C2=C2+ABS(H(L)-D2)
450 S3=S3+1
460 NEXT L
470 IF C2/S3<=T1 THEN 500
480 NEXT K
500 FOR K=1 TO P ! NEW FLOWS
510 S4=Q(K)
520 Q(K)=C(K)/ABS(Q(K))*H(J1(K)
)-H(J2(K))+.01)
530 IF I=1 THEN 550
540 Q(K)=.5*(Q(K)+S4)
550 NEXT K
560 C3=0 ! TOLERANCE CHECK
570 FOR K=1 TO P
580 C3=C3+ABS(F(K)-ABS(H(J1(K))-
H(J2(K)))/Q(K)-2*C(K)*F(K))
590 NEXT K
600 IF C3/P<=T0 THEN 620
610 NEXT I
620 FOR L=1 TO J3
621 H(L)=H(L)-Q2(L)*T5/A(L)
622 FOR M3=1 TO M1(L)
623 M=M2(L,M3)
624 IF J1(M)<>L THEN 626
625 H(L)=H(L)-Q(M)*T5/A(L)
626 IF J2(M)<>L THEN 628
627 H(L)=H(L)+Q(M)*T5/A(L)
628 NEXT M3
630 NEXT L
632 PRINT USING "K,DDDDD,X,K,DDD
DD.DD" ; "Ts=",T6,"H1=",H(1)
638 PRINT "TOPN BOT Xm Dm Qm
3/s HBOTm"
640 FOR K=1 TO P
650 PRINT USING 670 ; J1(K),J2(K)
,X(K),D(K),Q(K),H(J2(K))
660 NEXT K
670 IMAGE DDD,DDD,DDDDD,DD.DDD,D
DD.DDD,DDDDD.D
675 NEXT T6
680 STOP
690 END

```

NAME OF NETWORK?
 STORMDRAIN
 DRAWOFF DURN,min,SIM DURN,min,DT
 ,min,DRAWOFF1 m3/s?
 28,60,5,-1.4
 NPIPES,NODES,NRES,DARCY?
 4,5,4,.014
 INITL WATER LEVELm,SURFACE AREA
 m2 1 ?
 100,400
 2 ?
 100,400
 3 ?
 100,400
 4 ?
 100,100000
 TOPN,BOTN,Lm,Dm,OUTFLOBM3s 1 ?
 1,3,900,.8,-1.4
 2 ?
 3,2,1200,.7,-1.4
 3 ?
 3,5,800,1,-1.4
 4 ?
 5,4,1200,1,0

PIPENET STORMDRAIN
 Ts= 300 H1= 101.01
 TOPN BOT Xm Dm Qm3/s HBOTm
 1 3 900 .800 .056 101.6
 3 2 1200 .700 .035 101.1
 3 5 800 1.000 -.763 100.5
 5 4 1200 1.000 .634 100.0
 Ts= 600 H1= 102.39
 TOPN BOT Xm Dm Qm3/s HBOTm
 1 3 900 .800 -.442 102.3
 3 2 1200 .700 .263 102.3
 3 5 800 1.000 -.253 101.7
 5 4 1200 1.000 1.111 100.0
 Ts= 900 H1= 103.35
 TOPN BOT Xm Dm Qm3/s HBOTm
 1 3 900 .800 .125 103.4
 3 2 1200 .700 .066 103.4
 3 5 800 1.000 .093 102.4
 5 4 1200 1.000 1.303 100.0
 Ts= 1200 H1= 104.45
 TOPN BOT Xm Dm Qm3/s HBOTm
 1 3 900 .800 -.077 104.3
 3 2 1200 .700 -.068 104.4
 3 5 800 1.000 .201 103.3
 5 4 1200 1.000 1.552 100.0
 Ts= 1500 H1= 104.27
 TOPN BOT Xm Dm Qm3/s HBOTm
 1 3 900 .800 .249 103.5
 3 2 1200 .700 -.132 104.3
 3 5 800 1.000 1.360 102.6
 5 4 1200 1.000 1.357 100.0
 Ts= 1800 H1= 103.90
 TOPN BOT Xm Dm Qm3/s HBOTm
 1 3 900 .800 .484 103.2
 3 2 1200 .700 -.307 104.1
 3 5 800 1.000 1.237 102.1
 5 4 1200 1.000 1.234 100.0

Ts= 2100 H1= 103.55
 TOPN BOT Xm Dm Qm3/s HBOTm
 1 3 900 .800 .475 102.9
 3 2 1200 .700 -.327 103.8
 3 5 800 1.000 1.177 101.9
 5 4 1200 1.000 1.174 100.0
 Ts= 2400 H1= 103.21
 TOPN BOT Xm Dm Qm3/s HBOTm
 1 3 900 .800 .449 102.7
 3 2 1200 .700 -.334 103.6
 3 5 800 1.000 1.123 101.8
 5 4 1200 1.000 1.120 100.0
 Ts= 2700 H1= 102.90
 TOPN BOT Xm Dm Qm3/s HBOTm
 1 3 900 .800 .420 102.4
 3 2 1200 .700 -.335 103.3
 3 5 800 1.000 1.072 101.6
 5 4 1200 1.000 1.069 100.0
 Ts= 3000 H1= 102.60
 TOPN BOT Xm Dm Qm3/s HBOTm
 1 3 900 .800 .390 102.2
 3 2 1200 .700 -.332 103.1
 3 5 800 1.000 1.022 101.5
 5 4 1200 1.000 1.019 100.0
 Ts= 3300 H1= 102.33
 TOPN BOT Xm Dm Qm3/s HBOTm
 1 3 900 .800 .362 102.0
 3 2 1200 .700 -.328 102.9
 3 5 800 1.000 .973 101.3
 5 4 1200 1.000 .969 100.0
 Ts= 3600 H1= 102.08
 TOPN BOT Xm Dm Qm3/s HBOTm
 1 3 900 .800 .335 101.8
 3 2 1200 .700 -.322 102.6
 3 5 800 1.000 .924 101.2
 5 4 1200 1.000 .920 100.0

THE NEED TO VALIDATE HYDROLOGIC MODELS

by

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ABSTRACT

Hydrologic models can yield vastly different results depending on the model used and the experience of the user. Several models were utilized in estimating discharge values for storms of different frequencies in several watersheds in Prince George's County, Maryland. The estimates were produced independently by unrelated study groups.

The studies reveal the differences that can exist between values derived for the same stream segment using different models, and point out the critical need to assess the reasonableness of estimates from a hydrologic model before the employment of an estimate for design or regulatory purposes.

1.0 INTRODUCTION

A hydrologic model is a tool that can be used to assess a watershed event, such as a flood or drought. During or after a flood, such a model could be utilized in tracking the behavioral characteristics of the flood wave as it travels along the stream. How and where the flood waters overflow the banks could be noted, so also could the height of the flood waves. Such information is extremely important to a community as a first step in understanding its susceptibility to flooding and the magnitude and aerial extent of the problem. This initial step of understanding or appreciating the problem is necessary prior to undertaking a meaningful corrective action. It is also possible with flow records of sufficient length, for a community to gain some insight into future flooding problems and therefore take some preventive measures or actions to minimize prospective damages. A model could also be used to monitor the limited flow in a watershed during a drought. The fluctuations in flow along the stream and its tributaries between flood and drought

periods would provide invaluable information to the community on the stream's ability to meet its water resources needs at all times.

This paper presents case studies of flood estimates obtained by using different hydrologic models and reveals the vast differences that can exist between the estimates. The studies were all performed for watersheds located in Prince George's County, Maryland and were part of a county-wide assessment of flooding and drainage issues.

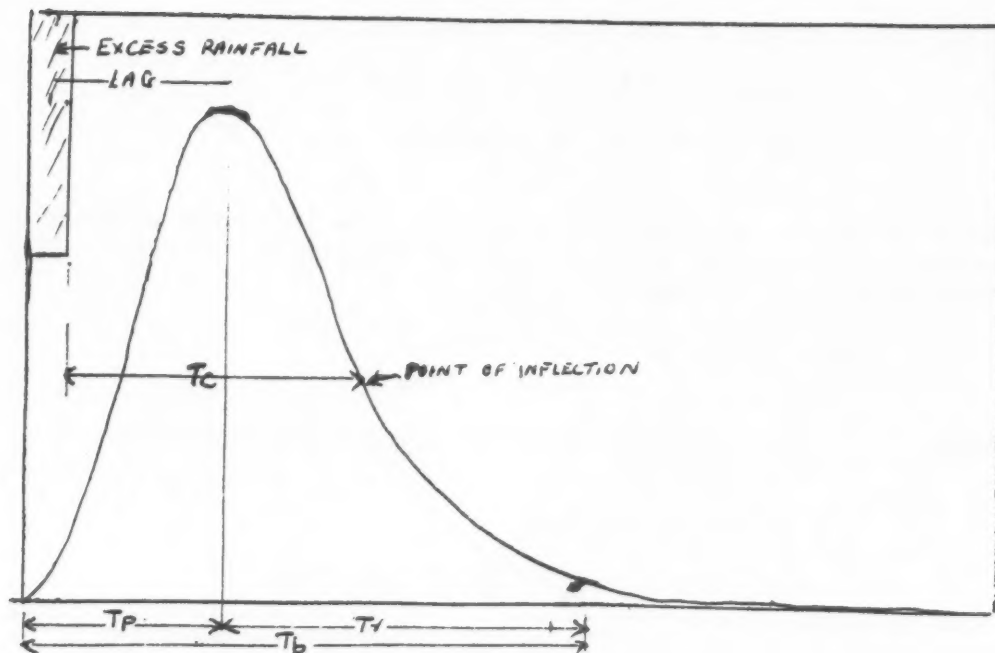
2.0 HYDROLOGIC MODELS

Hydrologic models are used to simulate hydrologic events that can be considered as systems or processes (Reference 1). The models considered in the paper are mathematical formulations used to describe a watershed's hydrologic and hydraulic phenomena. These models can be used to estimate or chart the runoff contributions of subareas, the aggregation of these contributions at various points within the watershed and the effect of obstructions, depressions and other flow features. Typical results from these models include discharge values, runoff volumes, hydrographs, hydrograph peaks and their times of occurrence. Some of the widely used hydrologic models are discussed here. Technical Release (TR) 20, developed by the Soil Conservation Service (Reference 2), is a single event computer program that computes runoff directly from natural or synthetic rainfall by applying runoff curve numbers to rainfall amounts. A runoff curve number (RCN) is a number which represents the watershed's potential to transform rainfall into runoff by considering the soil types and groups and the soil cover complexes in the watershed. Soil cover complexes are descriptions of the combinational effect of land use and land use treatment such as rural agricultural with vegetative cover and conservation practices, and the soil groups refer to the four soil group classifications (A, B, C, D) as defined by the Soil Conservation Service (Reference 3). Soil group A describes soils consisting chiefly of sands and gravels with low runoff potential while soil group D describes soils with high runoff potential. B and C group soils represent soils with moderate runoff potential.

Runoff curve numbers can range from a low of 6 for a pasture or range in good condition with contoured treatment to a high of 98 with the lower numbers representing low runoff yield and the higher numbers, high runoff yield. TR-20 uses the CN method to compute runoff and develop flood hydrographs by using the unit hydrograph addition procedure. The unit hydrograph in the program is a dimensionless curvilinear unit hydrograph that has a point of inflection approximately 1.70 times the time-to-peak (T_p) and the time to peak 0.2 of the time-of-base (T_b).

"HYTAIN" (Reference 4) is also a single event computer program for computing runoff from natural or synthetic rainfall. It was developed by the engineering firm of Clark, Finefrock and Sackett.

Dimensionless Curvilinear Unit Hydrograph



"Log-Pearson Type III" (Reference 5) is a statistical model that is used to estimate peak flows by fitting a Pearson-Type III distribution to a logarithm transformation of annual peak discharges at a gauge site. The mechanics involved in this method requires the computation of three parameters: the mean ($\bar{x}$), standard deviation (s), and coefficient of skew (g) of the logarithms of the values. The mean ($\bar{x}$), standard deviations (s), and the coefficient of skew (g) are derived using the following equations:

$$\bar{x} = \frac{\sum x}{n} \quad (3)$$

$$s = \left[\frac{\sum (x - \bar{x})^2}{(n-1)} \right]^{0.5} \quad (4)$$

$$g = \frac{n \sum (x - \bar{x})^3}{(n-1)(n-2)s^3}$$

in which:

$\bar{x}$ = logarithm mean

x = logarithm of annual peak flow

n = number of peak flows in data set

s = standard deviation of logarithms

g = skew coefficient of logarithms

These parameters are then used to calculate the logarithms of the peak values corresponding to the required recurrence interval. The equation used in the computation is:

$$\text{Log } Q = \bar{x} + KS$$

where Q = discharge value for the required recurrence interval

$\bar{x}$ = logarithm mean

S = standard deviation of logarithms

K = a Pearson Type III coordinate corresponding to the required interval and the computed coefficient of skew (g) (Reference 5)

Regression models are mathematical formulations used to derive flood discharges from variables such as meteorology, geology, topography or a combination of these. The combination of these variables, termed the independent variables and how they affect the flow discharge (the dependent variable) is usually established by a technique called multiple regression analysis (Reference 6). Some of the more popular regression models are: (a) the Maryland Geological Survey regression model (Reference 7). This model is of the form:

$$Q = A^i B^j z^k y^l E^m F^n$$

where Q = peak flow discharge in cfs (the dependent variable)

A, B, C, D, E, F = the independent variables representing the meteorologic, geologic, topographic characteristics of the drainage area.

i, j, k, l, m, n = the regression exponents

z, y = regression constants

Through a stepwise procedure, those independent variables, their constants and exponents which best estimate the dependent variable are established.

A summary of the regression relations is shown in Table 1.

TABLE 1
MARYLAND GEOLOGIC SURVEY REGRESSION MODEL

(Model: $P_n = KA^a S^b F^c G^d$)

Recurrence interval (n), in years, for flood peaks (P)	Regression constant K	Regression exponents for indicated basin characteristics				Standard error ± %
		a Area (A)	b Slope(S)	c Forest(F)	d Geography(G)	
2	54.2	0.947	0.331	-0.394	0.809	31.7
	60.7	.945	.339	- .428	0	36.2
	17.1	.913	.274	0	0	42.9
5	88.9	.921	.329	- .362	.856	30.5
	170	.920	.338	- .398	0	35.1
	30.8	.890	.270	0	0	41.3
10	112	.908	.336	- .337	.956	32.4
	41.5	.883	.284	0	1.05	37.0
	42.0	.878	.288	0	0	43.2
25	141	.894	.350	- .302	1.11	37.4
	57.5	.871	.303	0	1.20	40.8
	58.3	.865	.308	0	0	48.2
50	46.0	.915	.377	0	.909	36.7
	42.8	.911	.410	0	0	41.3
100 *	84.9	.874	.350	0	1.25	38.4
	87.1	.858	.358	0	0	46.2

* Relations not valid for Coastal Plain drainage basins.

(b) The Udhiri and Motayed regression model is of the form:

$$Q_n = D^i S^j S_t^k$$

D = drainage area of the watershed in square miles,
S = main channel slope in ft/mile and

S_t = surface storage in percent.
 i, j, k = regression exponents

A summary of the regression relations is presented in Table 2.

TABLE 2

UDHIRI AND MOTAYED REGRESSION MODEL

$$Q_2 = 23.0 D^{.873} S^{.291} S_{st}^{-.205} \quad ..(1)$$

$$Q_{10} = 75.7 D^{.814} S^{.266} S_{st}^{-.419} \quad ..(2)$$

$$Q_{50} = 188.4 D^{.765} S^{.243} S_{st}^{-.514} \quad ..(3)$$

$$Q_{100} = 275.4 D^{.742} S^{.229} S_{st}^{-.588} \quad ..(4)$$

where Q_2 = peak discharge in cfs for a 2-year recurrence flood

D = drainage area of the watershed in square miles

S_{st} = surface storage in percent.

3.0 PISCATAWAY CREEK FLOOD ESTIMATES

Some of the various hydrologic models discussed previously were used to estimate flood flows for different recurrence intervals at the same point on Piscataway Creek. The Creek is a tributary of the Potomac River and is located in the southwest portion of Prince George's County, within the Atlantic Coastal Plain physiographic province in Maryland (Figure 1). The Creek flows in a southwest direction for approximately 20 miles and empties into the Potomac River, some seven miles south of Washington, D.C. Piscataway Creek rises within the Andrews Air Force Base just north of South Perimeter Road at elevation 260 feet mean sea level and drains a total area of 62.8 square miles.

The soils in this area consist of silty, sandy and gravelly Pleistocene outwash from unconsolidated materials of the Coastal Plain. The predominant soils in the area belong to soil group C which defines soils with slow infiltration rates when thoroughly wetted (Reference 3). Most of the watershed is undeveloped although extensive development has taken place in the northernmost portion of the watershed. Overall, the watershed is approximately 17 percent developed.

The results of the flood estimates using some of the various models discussed in this paper are displayed in Table 3.

Figure 1



TABLE 3

COMPARISON OF PEAK FLOWS BY VARIOUS MODELS

PISCATAWAY CREEK TO PISCATAWAY ROAD, MD
DRAINAGE AREA - 39.5 square miles
100 YEAR PEAK FLOWS IN CFS

Frequency in years	TR-20 Model (Ref. 2)	Regression Model by Maryland Geological Survey (Ref. 7)	Regression Model by Udhiri & Motayed (Ref. 8)	Statistical Model (Log-Pearson Type III Distri- bution (Ref. 5))
100	8980	5228	7647	17746

4.0 ANACOSTIA RIVER FLOOD ESTIMATES

Estimates for floods for various recurrence intervals were made at a point (drainage area, 72.8 square miles) on the Anacostia River (Figure 2) which flows through two physiographic provinces (the Piedmont Plateau and the Atlantic Coastal Plain) in Montgomery and Prince George's Counties. The Piedmont Plateau is characterized by rocky channels with steep-sided valleys and is well-drained. On the other hand, the Coastal Plain has broad open valleys with streams that have flat slopes and shallow channels. On the Anacostia River, at the computational point, the U.S. Geological Survey has operated a stream gauge continuously since 1939. The flood estimates on the Coastal Plain portion of the Anacostia River at the computational point for various recurrence intervals using some of the various models discussed previously are shown in Table 4.

5.0 WESTERN BRANCH FLOOD ESTIMATES

Western Branch is located in the central portion of Prince George's County in Maryland (Figure 3). The stream is situated in the Atlantic Coastal Plain physiographic province, characterized by wide flood plains and shallow channels. Western Branch is formed by three tributaries, Bald Hill, Folly and Lottsford Branches. Folly and Lottsford Branches converge and shortly afterwards form a confluence with Bald Hill Branch. From this confluence, Western Branch follows a widening course along a flat stream gradient for approximately 16.5

miles before emptying its flow and those of several tributaries that drain into it, into the Patuxent River. With a total watershed area of 110 square miles, Western Branch drains approximately 22 percent of the County.

The predominant soil group in the area is soil group B which defines soils with moderate infiltration rates when thoroughly wet.

Figure 2

ANACOSTIA RIVER WATERSHED



TABLE 4

PEAK DISCHARGE VALUES OBTAINED BY USING DIFFERENT MODELSStation

Northeast Branch Anacostia at Riverdale Road, Maryland
(D.A. = 72.8 square miles)

Recurrence Intervals in years	Log- Pearson Type III Model	TR-20 Model	'HYTAIN' Model	USGS Model	MD. G.S. Regression Model	Udhiri & Motayed Model
2	2600	8342	9213	2418	1714	2027
5	4357	11450	12389	3640	2770	3200 +
10	5929	14258	14979	4758	3756	4854
25	8486	15286	16712	6240	5184	7200 +
50	10880	18195	18726	7410	6000	9559
100	13764	20751	21315	8500 *	8530	12263

+ Values obtained by interpolation

* Values obtained by extrapolation

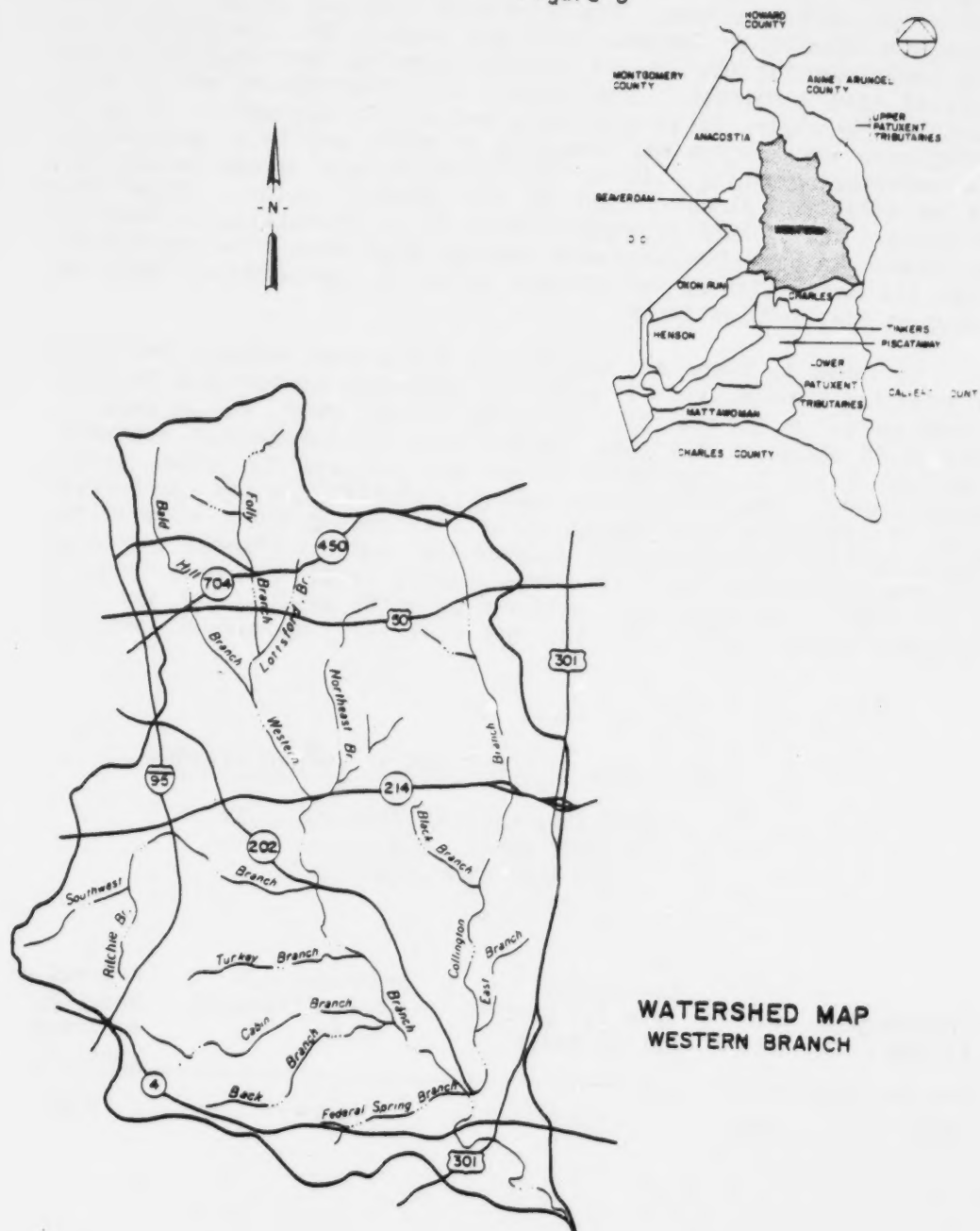
Development in the watershed is presently patchy with extensive development occurring inside the Capital Beltway (I-95) and sparse growth areas outside.

Estimates of flood peaks for various recurrence intervals using some of the models discussed previously are shown in Table 5.

6.0 DISCUSSION

Significant differences between flood peak estimates derived using different models become clearly evident in Tables 3, 4, and 5. At the same point on the Piscataway Creek (See Table 3), there is a 2419 cfs difference between the regression models and a 3752 cfs difference exists between the TR-20 value and the value from the Maryland Geological Survey regression model. However, the difference is only 1333 cfs between the TR-20 estimate and the "Udhiri and

Figure 3



Motayed" regression model. The Log-Pearson Type III model yielded an estimate of 17746 cfs whereas 8980 cfs was obtained using TR-20. The Log-Pearson Type II and TR-20 models are very widely used for hydrologic determinations. The Hydrologic Committee of the U.S. Water Resources Council in December 1967 and adopted the Log-Pearson Type III technique for use in all Federal planning involving water and related land resources. The Council also recommended use of this technique by state, local government and private engineers. TR-20 is being used with increasing frequency by state and local governments for watershed planning. It is also receiving wide spread recognition as an effective planning tool by the private sector. Given the widespread use of both of these models, it is interesting to note the difference in peak flow estimates derived from them. The Log-Pearson Type III model yielded an estimate which is approximately twice as large as the TR-20 estimate.

In Table 4, there are significant differences between the TR-20 and the Log-Pearson Type III peak flow estimates for the more frequent flood events (2, 5, and 10). Using the 10-year event, for an example, the TR-20 value is 2.4 times larger than the Log-Pearson estimate. For the 2 and 5, the TR-20 values are 3.2 and 2.6 times larger respectively. The differences between "HYTAIN" and the Log-Pearson Type III estimates are approximately the same as those with TR-20. There is some close agreement between the regression model estimates and the Log-Pearson Type III values especially for the more frequent flood events, although wide differences still exist. For the less frequent flood events (25, 50, and 100) the Log-Pearson Type III

TABLE 5
COMPARISON OF PEAK FLOWS BY VARIOUS METHODS

WESTERN BRANCH TO LARGO ROAD, MD
DRAINAGE AREA - 30.2 square miles
100 YEAR PEAK FLOWS IN CFS

Frequency in years	TR-20 Model	Log-Pearson Type III Distri- bution Model	Drainage-Area Discharge-Frequency Method	Regression Model by Anderson (Ref. 9)
100	7240	2411*	7977	7100

*Gauge was discontinued in 1972 because of debris problems which constantly affected gauge readings.

estimates are 8486, 10880 and 13764 cfs respectively. The TR-20 values for the same frequencies (25, 50, and 100) are 15286, 18195 and 20751 cfs, yielding differences.

The Western Branch 100-year peak estimates derived by the models in Table 5 show a surprising closeness. The TR-20 value for the 100-year flood at the Largo Road point (Drainage area of 30.2 square miles) is 7240 cfs. The regression model by "Anderson" (Reference 9) estimated the flow as 7100 cfs, and a 7977 cfs estimate was obtained from a Drainage Area-Discharge-Frequency model (Reference 10). The Log-Pearson Type III model estimate is 2411 cfs -- a value that is approximately one-third the size of the other estimates.

7.0 SUMMARY

The case studies presented herein have shown the differences that can exist between values derived using several hydrologic models. The differences in some instances are on the order of two to three. Hydrologic models can yield vastly different results depending on the model used and the experience of the user. These values determine to a large extent the size and configuration of measures to correct or prevent watershed problems. If the values obtained from a model are unreasonably large, their employment would result in uneconomical and wasteful facilities and their use for regulatory purposes would create undue burden and hardship on the community. If on the other hand, the values are unreasonably low, their employment in design would lead to undersized facilities with consequential results, and their use for regulatory purposes could have disastrous consequences.

On the basis of these case studies, there is a need, a critical need, to assess the reasonableness of flow estimates by hydrologic models. Such an assessment of estimates from a hydrologic model for reasonableness is oftentimes called validation.

REFERENCES AND BIBLIOGRAPHY

1. Chow, V.T., Handbook of Applied Hydrology, 1964.
2. Soil Conservation Service, Computer Program for Project Formulation, Hydrology, Technical Release No. 20, USDA, May 1965
3. Soil Conservation Service, National Engineering Handbook, Section 4, Hydrology, USDA, 1971.
4. Clark, Finefrock and Sackett, HYTAIN, Hydrology Model, 1976.
5. U.S. Water Resources Council, Guidelines for Determining Flood Flow Frequency, Bulletin 17B, September 1981.

6. Richmond, S.B., Statistical Analysis, 1964.
7. Maryland Geological Survey, Flow Characteristics of Maryland Streams, Report No. 16, 1971.
8. Udhiri, S., and Motayed, A., Magnitude.
9. Anderson, D.G., Effects of Urban Development on Floods in Northern Virginia, U.S. Geological Survey, Water Supply Paper 2001-C.
10. Udhiri, S., Review and Analysis of Storm Discharge Rates, Anacostia River Watershed, Technical Report Publication No. 0764791560, Maryland-National Capital Park and Planning Commission, September 1978.

MODELING OF A NEGATIVELY BUOYANT THERMAL DISCHARGE IN AN ESTUARINE ENVIRONMENT

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INTRODUCTION

Interest in thermal modeling stems from section 316(a) of the Water Pollution Control Act Amendments of 1972 (PL92-500) which required that limits be put on the thermal component of any discharge to "...assure the protection and propagation of a balanced, indigenous population of shellfish, fish, and wildlife in and on the body of water into which the discharge is to be made,..."

Several mathematical models have been developed over the last decade for predicting the behavior of thermal plumes. Since most cooling water discharges involved addition of heat to the circulating water, which results in the discharge of a buoyant or floating plume, most of the mathematical models available are applicable to floating plume conditions. There are some situations however, which result in the formation of negatively buoyant or sinking plumes. These include the discharge of relatively dense process waters, blowdown from cooling towers, and brine discharges.

Some experimental studies have been performed to characterize the behavior of negatively buoyant plumes but mathematical models are not readily available.

The structure of any discharge plume can be divided into near field and far field regions. In the near field inertial forces dominate gravitational forces, while in the far field gravitational forces dominate inertial forces. Mixing of a heated discharge with the ambient water in the near field is primarily due to inertia induced entrainment, while the natural transport processes of advection by ambient current and turbulent diffusion control mixing in the far field.

The densimetric Froude number being the ratio of inertial forces to gravitational forces, provides a convenient definition of the transition between the near and far fields. This transition is generally accepted as the location in the plume where the densimetric Froude number is unity.

Near Field Conceptual Model

Figures 1a and 1b illustrate the behavior of a surface discharge of a floating plume in the near field. Figure 1c illustrates the near field region of a sinking plume discharged at the bottom of the receiving water body. A desk top methodology has been developed by Jirka et al. (1981) which describes the near field behavior of a buoyant surface jet discharged into shallow water, as a function of densimetric Froude number (Figure 1b).

The primary variables of interest are the discharge length scale, l_o , and the densimetric Froude number, F_o . The discharge length, l_o , is defined as the square root of one half of the discharge cross-sectional area. In terms of a circular pipe l_o is:

$$l_o = \sqrt{\frac{D_o^2 \cdot \pi}{8}} \quad (\text{eq. 1})$$

where D_o = the pipe diameter (ft). The densimetric Froude number F_o is:

$$F_o' = \frac{U_o}{\sqrt{\frac{g(\rho_o - \rho_a)}{\rho_a} l_o}} \quad (\text{eq. 2})$$

where U_o = the discharge velocity (ft/s), g = gravitational acceleration (ft/sec²), ρ_o = discharge density (gm/ml), ρ_a = density of ambient receiving water (gm/ml).

Densities can be calculated as a function of salinity and temperature from the following expression:

$$\rho = 0.9997 - 0.0000063(T-4)^2 + 0.0007615(S) \quad (\text{eq. 3})$$

where T = temperature ($^{\circ}\text{C}$) and S = salinity (PPT).

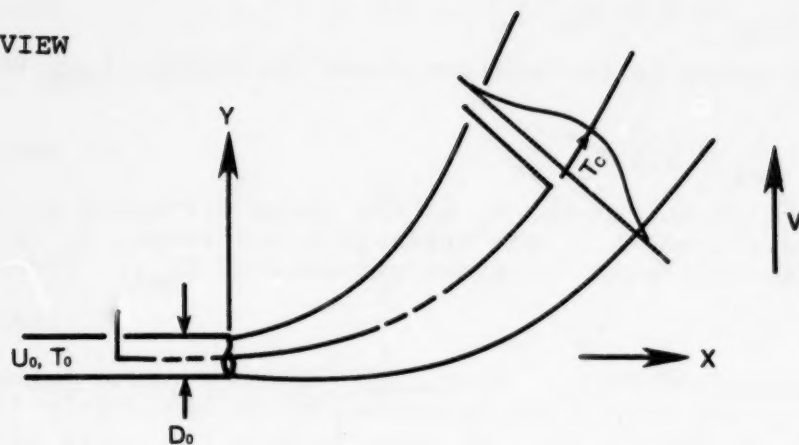
Jirka et al. (1981) compared laboratory and field data against simulations using the surface jet model of Stolzenbach and Harleman (1971) to develop a number of correlations between bulk near field properties of a surface jet and the variables F_o and l_o . These properties are described briefly below and illustrated by Figure 2b.

Transition distance, l_t is the extent of the near field (Figure 1b) and can be calculated as:

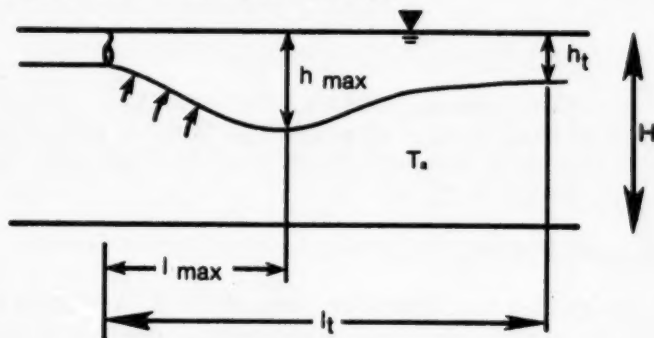
$$l_t = 10 F_o' l_o \quad (\text{eq. 4})$$

Maximum plume thickness, $h_{\max}$ (Figure 1b) is the maximum vertical penetration of the plume in the near field and can be calculated as:

A) PLAN VIEW



B) SURFACE DISCHARGE OF FLOATING PLUME



C) BOTTOM DISCHARGE OF SINKING PLUME

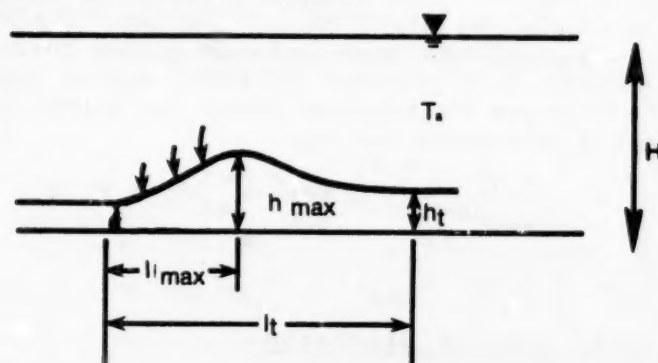


FIGURE 1 NEAR FIELD PROFILES OF BUOYANT SURFACE AND NEGATIVELY BUOYANT BOTTOM DISCHARGES

$$h_{\max} = 0.42 F_o' l_o \quad (\text{eq. 5})$$

The distance to the maximum plume thickness, $l_{\max}$ is calculated as:

$$l_{\max} = 5.5 F_o' l_o \quad (\text{eq. 6})$$

Transition thickness, h_t is the plume thickness at the transition distance, l_t . The transition thickness, h_t is nearly constant and equal to about one-half of $h_{\max}$. Thus:

$$h_t = 0.21 F_o' l_o \quad (\text{eq. 7})$$

Stable bulk dilution, S_t is the ratio of plume volumetric flow at the transition distance, Q_t to volumetric flow rate at the discharge, Q_o . S_t also denotes the ratio of the jet-average excess temperature (or concentration) at the transition distance to the excess temperature (or concentration) of the discharge. This dilution is given by:

$$S_t = 1.4 F_o' \quad (\text{eq. 8})$$

Stable centerline dilution, S_{tc} is the ratio of the centerline temperature (or concentration) excess at the transition distance to the temperature (or concentration) excess at the discharge and is given by:

$$S_{tc} = 1.0 F_o' \quad (\text{eq. 9})$$

The properties defined by equations 4 through 9 above are for surface discharges into stagnant, deep receiving waters. In shallow water a buoyant surface discharge will "touch the bottom" and a negatively buoyant bottom discharge will "touch the surface." In either case vertical entrainment and mixing will be reduced. Jirka et al. (1981) studied the effect of shallow depths on dilution in the near field and concluded that dilution is reduced when maximum plume thickness is greater than seventy-five percent of total water depth. The ratio, r_s , of dilution at shallow water to dilution at deep water in the near field is given by:

$$r_s = \frac{0.75}{h_{\max}/H}^{0.75} \quad \text{for } h_{\max}/H > 0.75 \quad (\text{eq. 10})$$

$$\text{and } r_s = 1.0 \text{ for } h_{\max}/H \leq 0.75 \quad (\text{eq. 11})$$

Negatively Buoyant Discharge

While similar to an "inverted surface discharge," a negatively buoyant bottom discharge will experience additional friction

which will reduce the size of the near field and the resulting dilution. A few experiments have been performed with negatively buoyant bottom discharges. The results of one such experiment by Baddour and Chu (1978) were analyzed by Jirka et al. (1981) using the same dimensional analysis discussed above. In comparison with surface discharges the transition distance was observed to be reduced by about 50 percent while the bulk dilution was reduced by about 65 percent. Thus, for a negatively buoyant bottom discharge in deep water:

$$l_t = 5 F_o' l_o \quad (\text{eq. 12})$$

and

$$S_t = 0.5 F_o' \quad (\text{eq. 13})$$

In the case of shallow water the dilution given by equation 13 may be modified by equation 10.

FAR FIELD MODEL

There are many numerically solved models available to simulate far field transport behavior. The transient plume model, (TPM) developed by Adams et al., 1980 is an analytically solved far field model which was formulated for steady-state surface discharges of floating plumes. It is a three dimensional model which can simulate tidal variations in the receiving water. Conservative and non-conservative constituents with first order decay can be modeled. Dilution of the plume occurs by turbulent diffusion and advection by the ambient current is taken into account by the model. The TPM model assumes that receiving water depth is constant and velocity is considered to be spatially invariant, although temporal variations such as tidal cycles may be specified as a Fourier series function. Both along-shore and onshore-offshore components of velocity can be considered. The shoreline boundary is assumed to be straight.

Boundary conditions for the mode are obtained from the near field analysis, and consist of location, volume flux of the plume, and temperature and plume thickness at the end of the near field.

Transient heat distribution is computed as a series of discrete overlapping "puffs" (Figure 2a). Heat balance is conserved within each puff while dispersion increases the size of each puff with time. Temperatures within each puff are distributed normally with maximum temperatures at the center of each puff. Figure 2b is the far field region of a conceptualized plume in terms of temperature excess isotherms.

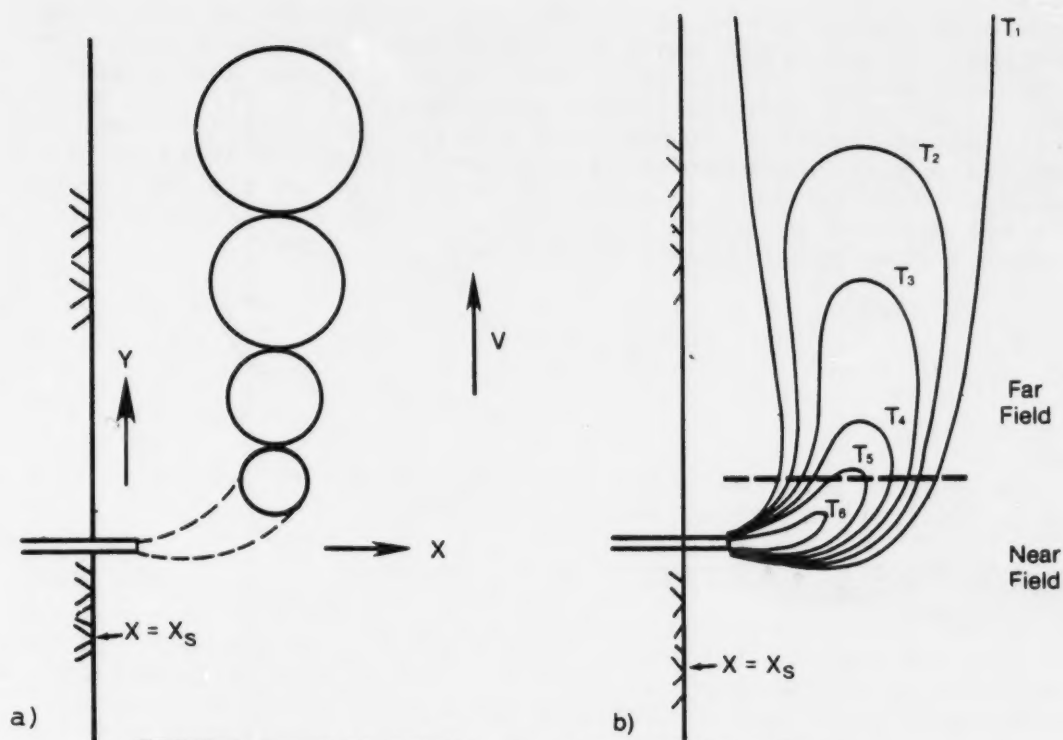


FIGURE 2 PLAN VIEWS OF A FAR FIELD PLUME

As with the near field analysis, the similarity between a floating surface plume and a sinking bottom plume makes it possible to apply this model to negatively buoyant plumes. This is accomplished by "inverting the Z axis" of the coordinate system. For a floating surface plume the depth coordinate, is normally considered to be zero at the surface and increases with depth, however by using $Z = 0$ to be the bottom, and $Z = H$ to be the surface, a dense submerged plume can be simulated. This is illustrated in Figure 3. Specific input requirements for the TPM may be found in the user's manual (Adams et al., 1980).

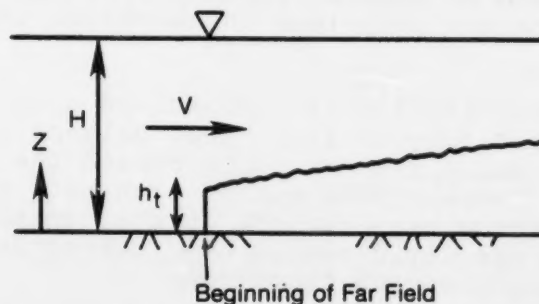


FIGURE 3 PROFILE OF A DENSE SUBMERGED PLUME

CASE STUDY

A thermal modeling study was performed to determine the seasonal mixing zone characteristics and temperature increase resulting from the interaction of the Proposed Hope Creek Generating Station (HCGS) heat dissipation system and the Delaware River, (Deb and Snyder, 1982).

The HCGS is located adjacent to the Salem Nuclear Generating Station (SNGS) on Artificial Island, a three-mile long strip of land on the east bank of the Delaware River in Lower Alloways Creek Township, Salem County, New Jersey.

HCGS construction began in March 1976 and Unit 1 operation, producing approximately 1.067 megawatts of electric power, is scheduled for 1986.

Cooling tower blowdown will be discharged to the Delaware river through a 48-inch diameter pipe, extending approximately 10 feet offshore at mean tide and approximately 1,500 feet upriver of the station intake. The center line of the opening will be about 6 feet below low mean water. The discharge will be at a velocity of approximately 4.08 feet per second. The layout of the Proposed cooling tower blowdown discharge is illustrated in Figure 4.

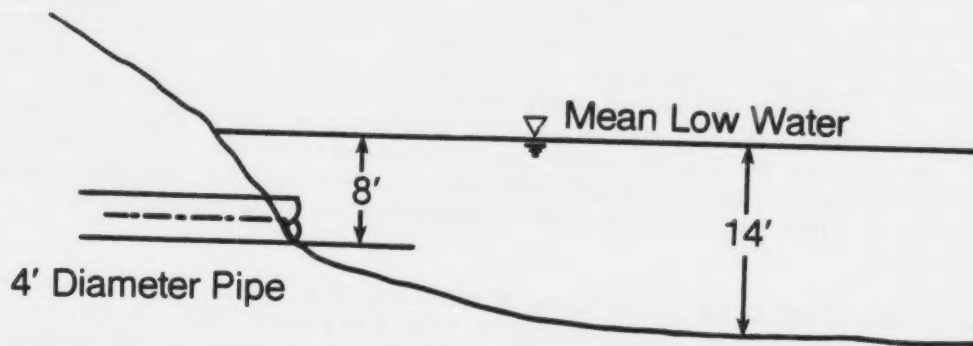


FIGURE 4
LAYOUT OF COOLING WATER DISCHARGE

Temperature standards for Zone 5 of the Delaware River have been established by the Delaware River Basin Commission, DRBC (1978). These regulations require that:

"The induced increase above ambient temperature shall not exceed 4°F (2.2°C) from September to May and 1.5°F (0.8°C) from June to August, or a maximum of 86°F (30.0°C), whichever is less, which temperatures shall be measured outside of designated heat dissipation areas."

"As a guideline, heat dissipation areas shall not be longer than 3,500 feet, measured from the point where the waste discharge enters the stream."

The objective of the study was to determine whether these temperature standards will be met after the proposed cooling tower blowdown discharge is put into operation.

Salinity concentration factors for the cooling tower are estimated to range from 1.42 to 1.68 under normal operating conditions. Under "worst case" conditions, which are expected to occur less than 5 percent of the time, the salinity concentration factor is 1.80.

The months of February, May, August and November were modeled to simulate the range of seasonal conditions using appropriate ambient temperature and salinity data. Each of these 4 months was modeled under "typical" and "worst case" operating conditions using the operating parameters which are presented in Table 1. The near field analysis indicated that the proposed HCGS discharge would be negatively buoyant under all seasonal and operating conditions except during May under "worst case" operating conditions when the discharge will be positively buoyant.

Eight computer runs were made with the TPM using the results from the near field analysis for the seasonal and operating conditions described above. Each computer run simulated conditions at the four tidal phases (i.e., low slack, flood, high slack, and ebb tides).

In order to visualize the three dimensional nature of the discharge plume, temperature profiles were printed out at the surface, mid-depth, and the bottom of the estuary for each seasonal, operating, and tidal condition. Excess temperature isotherms have been plotted for all runs. Figures 5 to 9 show plots of some of the excess temperature isotherms.

RESULTS AND DISCUSSION

The results of this analysis indicated that the DRBC temperature regulations of 4°F (2.2°C) from September to May and 1.5°F (0.8°C) from June to August or a maximum of 86°F (30°C) outside of a 3500 feet mixing zone will be met even under worst case conditions.

The largest mixing zone distance required for conformance with standards is 2230 feet at the estuary bottom (Figure 5) during the February worst case condition at high slack tide, well within the DRBC 3500 feet distance. A temperature increase of 4°F (2.2°C) at the water surface during these same conditions occurs within 600 feet of the HCGS discharge (Figure 6). The predicted temperature excess profiles for the month of February at high slack tide under worst case operating conditions at the bottom of the estuary and at the water surface are shown in Figures 5 and 6 respectively. The predicted temperature excess profile at the estuary bottom for the month of February during the same worst case operating conditions at ebb, low slack and flood tides are shown in Figures 7, 8, and 9 respectively.

The least critical month was the May typical condition, where standards are met 500 feet from the discharge.

Maximum temperature in the month of August at the end of the 3500 feet mixing zone will be 80.4°F (26.9°C) which is within the limit of the 86°F (30°C) DRBC standard.

This study clearly indicated that HCGS discharge will meet all DRBC temperature standards, even under "worst case" conditions.

Summary and Conclusions

A generalized near field and far field analysis applicable to negatively buoyant discharges was discussed. The analysis of a surface discharge of a floating plume was modified to account for bottom friction and surface interference for application to a bottom discharge of a sinking plume.

TABLE 1
AMBIENT AND DISCHARGE CONDITIONS

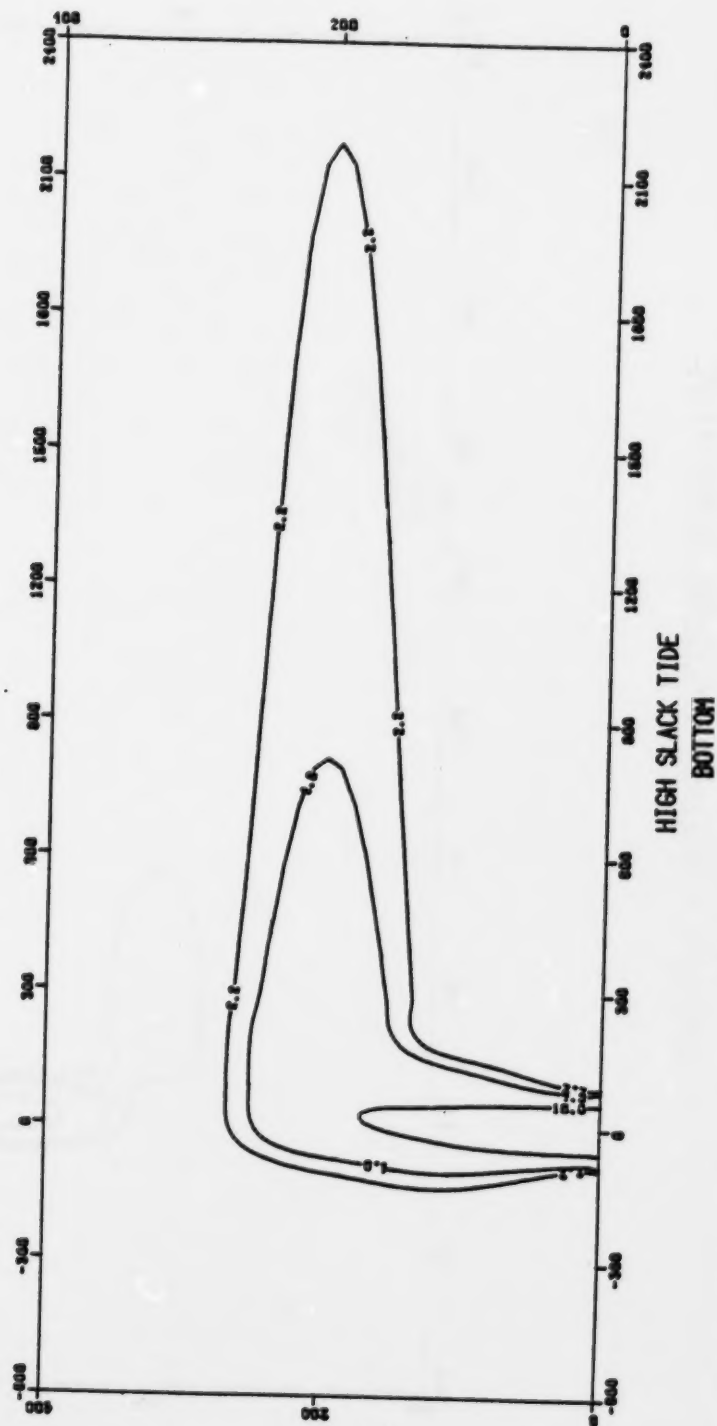
OPERATING CONDITIONS	AMBIENT RIVER CONDITIONS			DISCHARGE CONDITIONS			$\frac{\rho_o - \rho_a}{\rho_a}$	DENSIMETRIC FROUDE NO.
	TEMP. (°C)	SALINITY (PPT)	DENSITY (ρ_a)	TEMP. (°C)	CONC. FACTOR	SALINITY (PPT)	DENSITY (ρ_o)	
<u>TYPICAL CASE</u>								
February	1.4	4.25	1.00289	17.5	1.42	6.04	1.00315	0.00026 (n) [*] 28.1
May	18.9	3.15	1.00070	25.0	1.61	5.07	1.00078	0.00008 (n) 50.7
August	26.7	7.85	1.00243	29.0	1.68	13.19	1.00581	0.00337 (n) 7.8
November	9.5	6.10	1.00415	20.5	1.49	9.09	1.00491	0.00075 (n) 16.6
<u>WORST CASE</u>								
February	1.4	4.25	1.00289	21.0 ^{**}	1.80	7.65	1.00370	0.00081 (n) 15.9
May	18.9	3.15	1.00070	27.0 ^{**}	1.80	5.67	1.00069	0.000015 (p) 116.8
August	26.7	7.85	1.00243	31.0 ^{**}	1.80	14.13	1.00587	0.00343 (n) 7.7
November	9.5	6.10	1.00415	25.5 ^{**}	1.80	10.98	1.00515	0.00099 (n) 14.4

^{*}(n) = negatively buoyant

(p) = positively buoyant

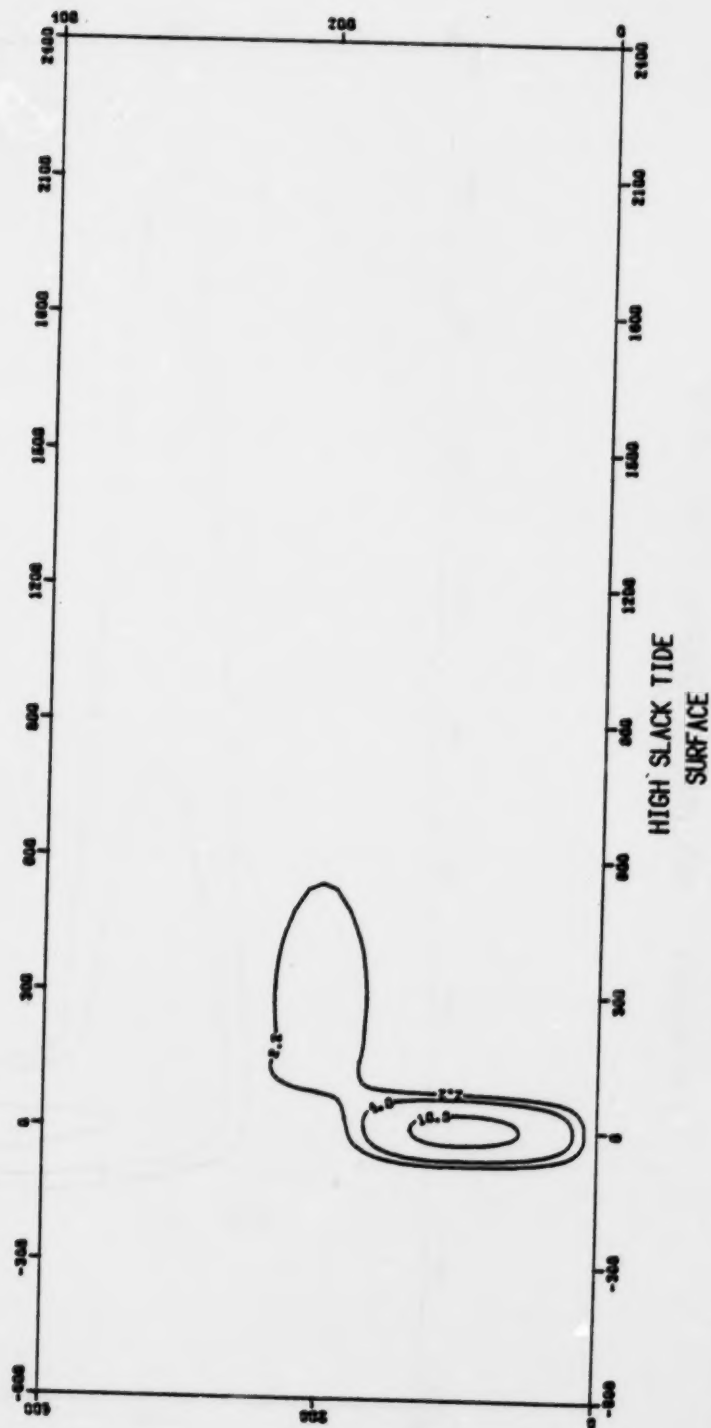
^{**} = Worst case, exceeded less than 5% of normal operation time

FIGURE 5
 TEMPERATURE (ΔT) ISOTHERMS ($^{\circ}\text{C}$)
 FEBRUARY - MAXIMUM DISCHARGE TEMPERATURE*



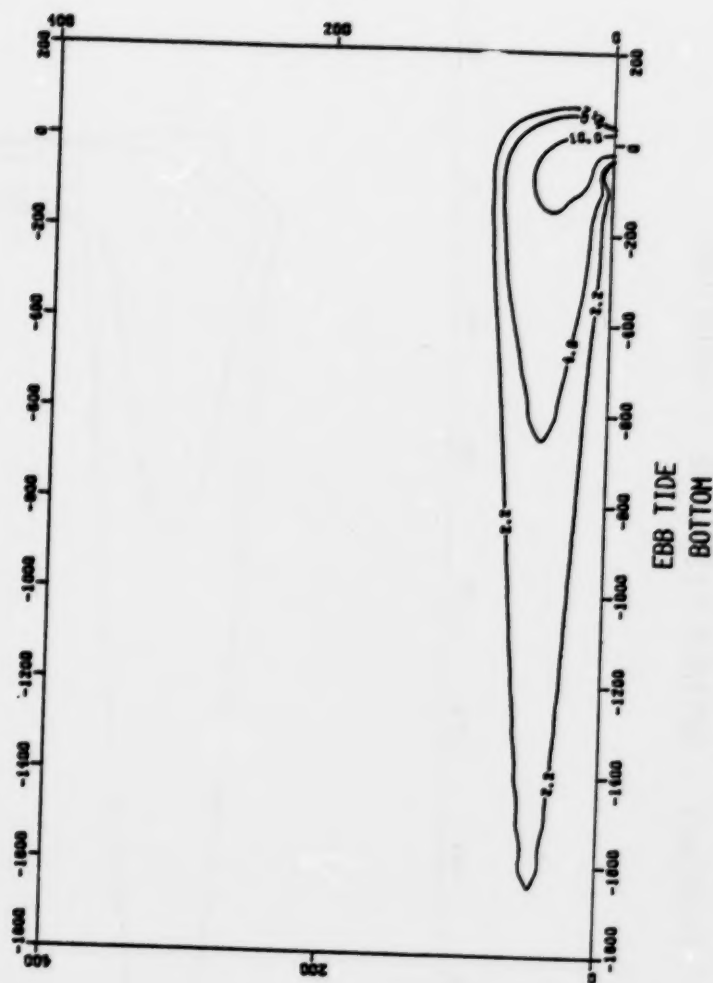
* EXPECTED TO BE EXCEEDED LESS THAN 5% OF THE TIME DURING NORMAL PLANT OPERATION

FIGURE 6
 TEMPERATURE (ΔT) ISOTHERMS ($^{\circ}\text{C}$)
 FEBRUARY - MAXIMUM DISCHARGE TEMPERATURE*



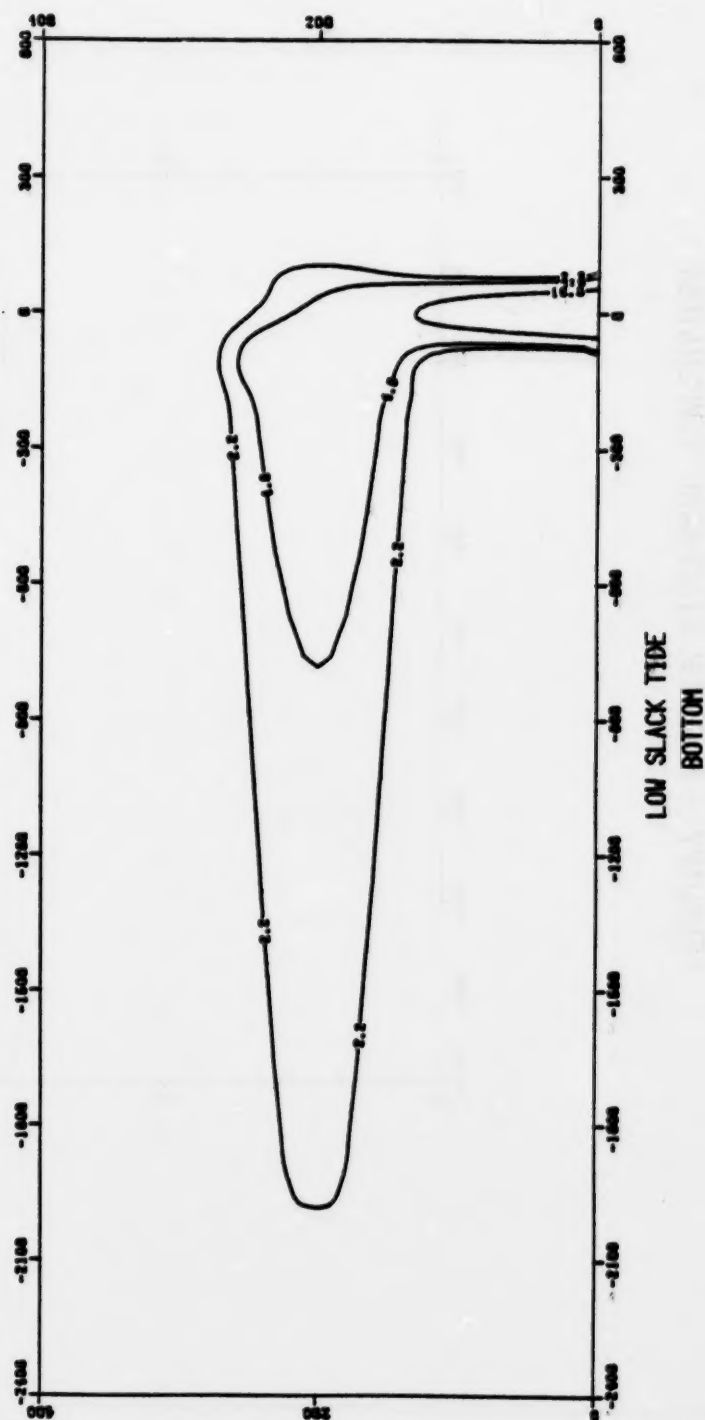
* EXPECTED TO BE EXCEEDED LESS THAN 5% OF THE TIME DURING NORMAL PLANT OPERATION

FIGURE 7
TEMPERATURE (ΔT) ISOTHERMS ($^{\circ}\text{C}$)
FEBRUARY - MAXIMUM DISCHARGE TEMPERATURE*



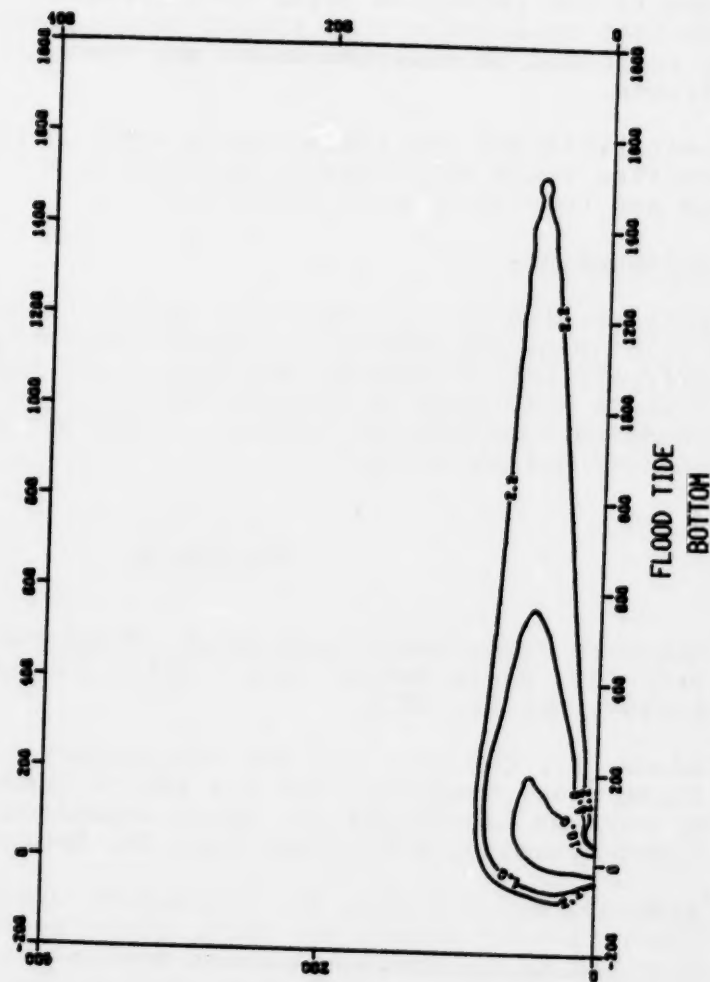
* EXPECTED TO BE EXCEEDED LESS THAN 5% OF THE TIME DURING NORMAL PLANT OPERATION

FIGURE 8
 TEMPERATURE (ΔT) ISOTHERMS ($^{\circ}\text{C}$)
 FEBRUARY - MAXIMUM DISCHARGE TEMPERATURE*



* EXPECTED TO BE EXCEEDED LESS THAN 5% OF THE TIME DURING NORMAL PLANT OPERATION

FIGURE 9
 TEMPERATURE (ΔT) ISOTHERMS ($^{\circ}\text{C}$)
 FEBRUARY - MAXIMUM DISCHARGE TEMPERATURE*



* EXPECTED TO BE EXCEEDED LESS THAN 5% OF THE TIME DURING NORMAL PLANT OPERATION

The methodology was applied to an actual case study involving the proposed discharge of blowdown water from a cooling tower into an estuarine receiving water. Predicted temperature increases in the receiving water were presented and it was concluded that applicable temperature regulations in the receiving water would not be violated under any seasonal and operating conditions.

The near field and far field models used in the case study are generalized tools which can be applied to a variety of discharge and receiving water conditions.

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REFERENCES

1. Delaware River Basin Commission, "Administrative Manual-Part III, Basin Regulations - Water Quality," Trenton, New Jersey, May 24, 1978.
2. Adams, E., Gaboury, D., and Stolzenbach, K., "Transient Plume Model Computer Code and User's Manual, Report, R. M. Parsons Laboratory for Water Resources and Hydro-dynamics, MIT, Cambridge, MA, September 1980.
3. Baddour, R., and Chu, V., "Turbulent Gravity - Stratified Shear Flows," Report No. 78-3, Fluid Mechanics Lab., Dept. of Civil Engineering and Applied Mathematics, McGill University, Montreal, Quebec, Canada, September 1978.
4. Jirka, G., Adams, E., and Stolzenbach, K., "Buoyant Surface Jets," Journal of Hydro. Division, ASCE, Volume 107, No. HY11, Nov. 1981, pp. 1467-1487.
5. Deb, A.K., and Snyder, J. K., "Thermal Modeling Study of Hope Creek Generating Station Discharge," Report Submitted to Public Services Electric and Gas Co., Newark, New Jersey, Roy F. Weston, Inc., West Chester, PA, October, 1982.

RAINFALL DATA ACQUISITION AND PROCESSING
USING APPLE II WORKALIKES

by

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ABSTRACT

Because of the spatial and temporal variability of thunderstorm type events, rainfall data from a network on a 2 km grid is essential to properly model urban stormwater systems. Conventional rainfall recording stations employing tipping bucket gauges with strip chart recorders suffer from inaccuracies in timing, mechanical failure, significant initial capital outlay and potentially large on-going data processing costs.

This paper describes a microcomputer-based data acquisition system, in successful operation in Hamilton since 1981, based on an inexpensive APPLE II workalike functioning as the main data processing facility for the system.

INTRODUCTION

It has been shown that correct representation of the spatial and temporal resolution of rainfall is essential for proper urban stormwater modelling (James and Scheckenberger, 1983). The density of the network of raingauges

monitoring an urban area will directly affect the quality of the data obtained. The larger the number of gauges the greater the potential for accurately describing storm speed, direction and cell size and hence for obtaining a proper estimate of rainfall intensity during a storm event. A network with a maximum spacing of about 2 km is essential if summer thunderstorm cells are to be detected.

Traditionally, rainfall monitoring stations have employed a tipping bucket rain gauge for collecting the rainfall and a mechanical strip chart recorder using either ink or pressure sensitive paper as the recording medium.

It has been our experience that tipping bucket gauges function reliably but are relatively expensive to purchase (approximately \$500 per instrument). At this price a large dense network is usually not feasible. Mechanical recorders have been found to be prone to frequent and significant errors in timing and synchroniztion due to gear malfunctions and basically insensitive equipment. Data is often lost due to paper jamming on drive sprockets and interruptions in ink flow to the recorder pens, for example due to air bubbles in the ink well. In Canada, tipping bucket recorders typically cost from \$300-\$600. Additionally, if pressure sensitive paper is required, the cost of continuously recording 4 to 5 months of data can exceed \$150/gauge. These costs add up to a fairly substantial initial investment per gauging location.

Moreover, the hidden cost in processing data from strip charts is high. The data must be abstracted, aggregated or disaggregated and subsequently entered into some storage facility, typically a computerized hydrologic data base, often manually, for each station. Our experience has shown this annual cost to be several times that of the initial capital cost.

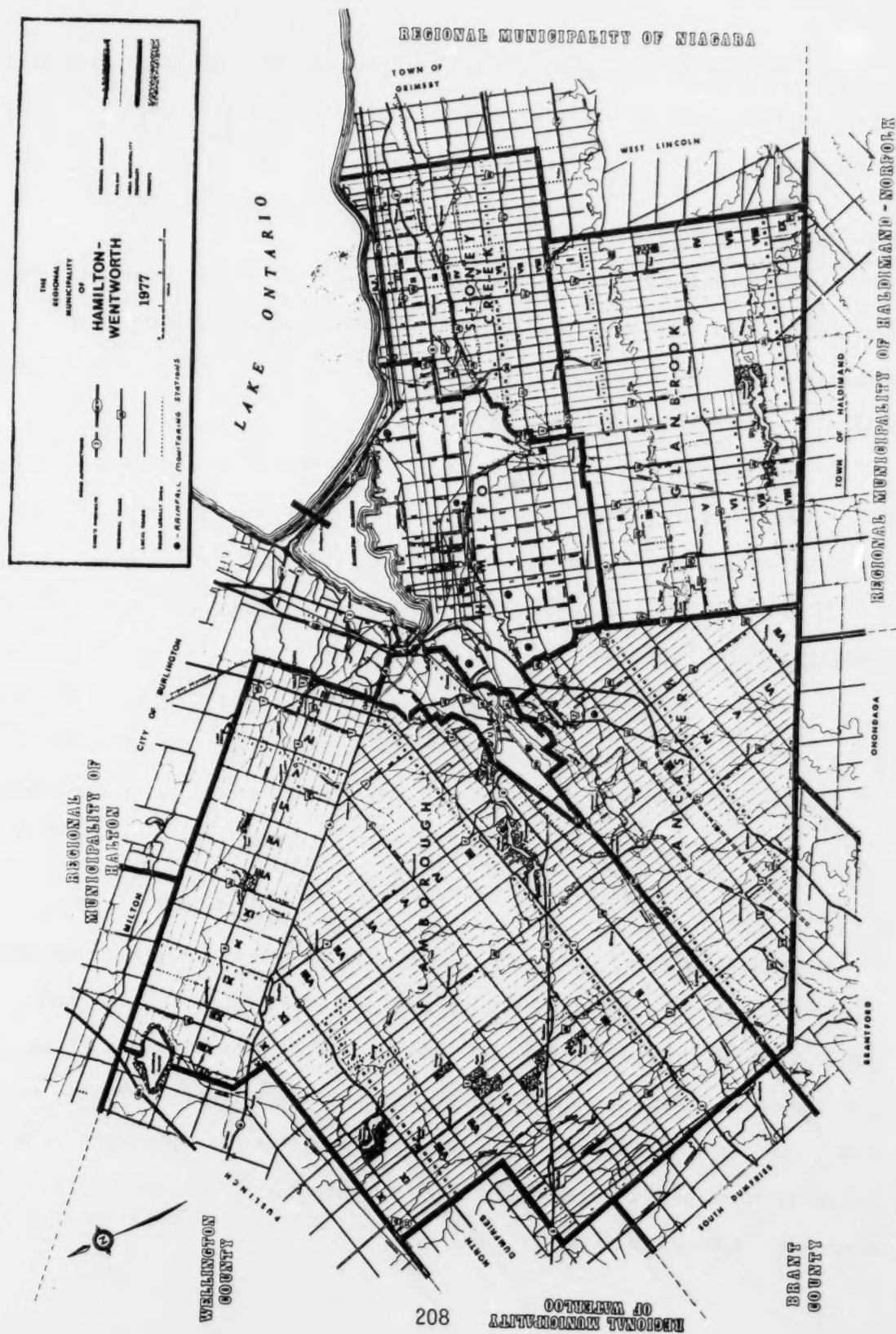
Our solution to this problem was to design and build our own inexpensive data acquisition system and to interface it directly with a central microcomputer.

RAINFALL DATA ACQUISITION SYSTEM.

The Computational Hydraulics Group at McMaster University has spent the past three years developing and implementing a microcomputer based data acquisition system (Haro et al, 1983; James et al, 1982). The data acquisition system has three principal components:

1. The rainfall sensor which collects precipitation and converts it into water drops of almost constant size to be counted by the data logger. This sensor can also be a conventional tipping bucket raingauge.
2. The data logger which senses the drops and counts them for a fixed time interval, typically one minute. The time is monitored by the system on a cycle from one to 240 minutes using its internal clock. The time and data are output by the logger to standard audio cassette tapes.
3. The data stored on cassette is in hexadecimal code. It is converted to ASCII code using a decoder. This data is output at a speed of 1200 baud through an RS232 serial communications port.

A rainfall monitoring network comprising nine stations has been operated successfully in the city of Hamilton since 1981 (see Figure 1). The data acquired during this period has been used to calibrate storm models (James and Scheckenberger, 1984) and models of the urban drainage system (Robinson and James, June 1984). With proper maintenance, the system has been found to be more reliable, efficient, accurate and less expensive than conventional data acquisition systems employed previously.



The data acquired by the system was initially processed and archived on a PDP 11/23 minicomputer with two 10-megabyte hard disks operating RSX-11M. However, as the system demand in terms of available terminals, memory and disk storage space increased, it was decided to move simpler tasks and word processing to peripheral computers such as Apple II workalikes, for example, the FRANKLIN ACE 1000.

The advantages of peripheral computing include the use of floppy diskettes as an archive media, an improvement over our current method of backup from the 11/23 to magnetic tape at low speeds (300 baud). It was subsequently also decided to implement the data processing utilities on the FRANKLIN ACE and use it for data capture in the hours when word processing was not being done.

DATA CAPTURE USING APPLE II WORKALIKES.

Personal microcomputers such as the Apple II and workalikes are well suited to data acquisition functions for the following reasons:

1. They are inexpensive and can be totally dedicated to a task.
2. Since data acquisition does not require large data arrays or high precision calculations, 8-bit word size with 48-64 Kbytes of memory will suffice.
3. Organization and archiving of files by station can be readily accomplished with floppy diskettes. This storage media is cost effective.
4. Built-in medium resolution graphics are available for data display.
5. Communication between the microcomputer and a larger computer containing say the central data base, is easily achieved.
6. The microcomputer can run some simulation packages using the processed data directly.

7. The microcomputers can be interfaced with peripheral devices such as printers inexpensively by purchasing an interface card, usually at a cost of about \$100.

In our system we have utilized the following hardware configuration:

1. FRANKLIN ACE 1000 microcomputer (APPLE II compatible)
2. 48 Kbytes RAM
3. DOS 3.3
4. One floppy disk drive
5. Grappler+ Printer Interface Card
6. 80 column text card
7. PDA232C Serial I/O Card
8. EPSON FX-100 printer

A tool kit of utilities has been developed, in APPLESOFT BASIC, for processing the rainfall data acquired by the system (Robinson and James, January 1984). Many of these utilities were originally written in FORTRAN for the PDP 11/23 and have been translated to APPLESOFT BASIC. The utilities have three primary component functions:

1. Decoding data
2. Translating data
3. Displaying (plotting) data

The programs are controlled by executive files which configure memory and load relevant programs into memory. A brief outline of the function of each component follows.

1. Decoding Data.

As described previously, the decoder outputs the data as a string of ASCII numbers to an RS232 port. This port is connected directly to the PDA232C Serial

I/O card. Data is transmitted from the decoder to a scratch file on the FRANKLIN ACE in blocks of 120 characters. The utility then disables input to the serial card. The utility further converts this data into 80 character records and stores it on diskette as a permanent file.

2. Translating Data.

The translation utilities process the file of decoded data into a series of chronological times and rainfall intensities. The processed data is displayed on the system monitor and optionally on the printer as well. Results may be calculated in either system of units. Volume and intensity on a per minute basis are output. Event volume and duration are displayed. The processed data is stored on diskette and may be structured so that it is in a form suitable for input directly to other packages such as a data base management system, a hydrologic model or graphics routines. A sample data processing session is presented in Figure 2.

3. Displaying Data.

The file of translated data containing one or more events may be processed at any time by the graphics utility. The user selects the event to be displayed either by start time or by its sequence in the file of events. The user selects whether or not the scaling of the vertical axis will depend on the peak intensity. Use of medium resolution graphics in conjunction with specially designed shape tables produce hyetographs, such as that of Figure 3. The Dialogue has been designed carefully (James and Robinson, 1982) and the next step will be to integrate the software into our evolving distributed data processing system (James and Unal, 1984).

FIGURE 2: SAMPLE DAS INTERPRETER SESSION

```

*****
DAS INTERPRETER UTILITY WRITTEN BY:
                                NAME ROBINSON, OCTOBER 1983
*****

ENTER NAME OF FILE TO BE TRANSLATED          TESTCASE3
ENTER NAME OF FILE TO CONTAIN TRANSLATED DATA  TEST32
ENTER GAUGE IDENTIFICATION                    TEST3
ENTER GAUGE UNIT NUMBER                        3
ENTER UNITS TO BE USED (METRIC/IMPERIAL)       METRIC
ENTER DATA SOURCE (CHARTROLL OR CASSETTE)     CASSETTE
ENTER STARTING TIME (YEAR,MO,DY,HR,MIN)       1982,9,21,16,30
ENTER OBSERVED TOTALIZER VOLUME (MM/IN)       30.0
ENTER TYPE OF GAUGE (D=DROP COUNTER, T=FLIPPING BUCKET) D
ENTER TYPE OF DROP COUNTER (B=BLACK, W=WHITE) B
ENTER INTER-EVENT PERIOD (MINUTES)            60
ENTER TIME-STEP FOR PLOTTING HYETOGRAPH (MINUTES) 5
PROCESSED RAINFALL DATA FOR 1982 FOR TEST3 UNIT NO. 3

                                RAINFALL                                EVENT
                                VOLUME INTENSITY                       VOLUME DURATION
                                (MM)  (MM/HR)                         (MM) (MIN)

9   21   16:33                   .1444    8.664
9   21   16:34                   .2128   12.768
9   21   16:35                   .3496   20.976
9   21   16:36                   .1596    9.576
9   21   16:37                   .152     9.12
9   21   16:38                   .1596    9.576
9   21   16:39                   .3268   19.608
9   21   16:40                   .1292    7.752
9   21   16:41                   .0988    5.928
9   21   16:42                   .228    13.68
9   21   16:43                   .4408   26.448

WARNING - ZERO RAINFALL DETECTED WITH NON-ZERO TIME
ACTION  - DATA WILL BE IGNORED
          CHECK SOURCE DATA FILE
WARNING - ZERO RAINFALL DETECTED WITH NON-ZERO TIME
ACTION  - DATA WILL BE IGNORED
          CHECK SOURCE DATA FILE

9   21   18:10                   .152     9.12
9   21   18:30                   .2432   14.592
9   21   18:50                   .152     9.12
9   21   19:10                   .2432   14.592
9   21   19:30                   .152     9.12
9   21   19:50                   .38     22.8
9   21   20:10                   .152     9.12
9   21   20:30                   0      0
9   21   20:35                   .152     9.12

                                2.4016   11
                                1.6264   146

```



FIGURE 2 (Continued)

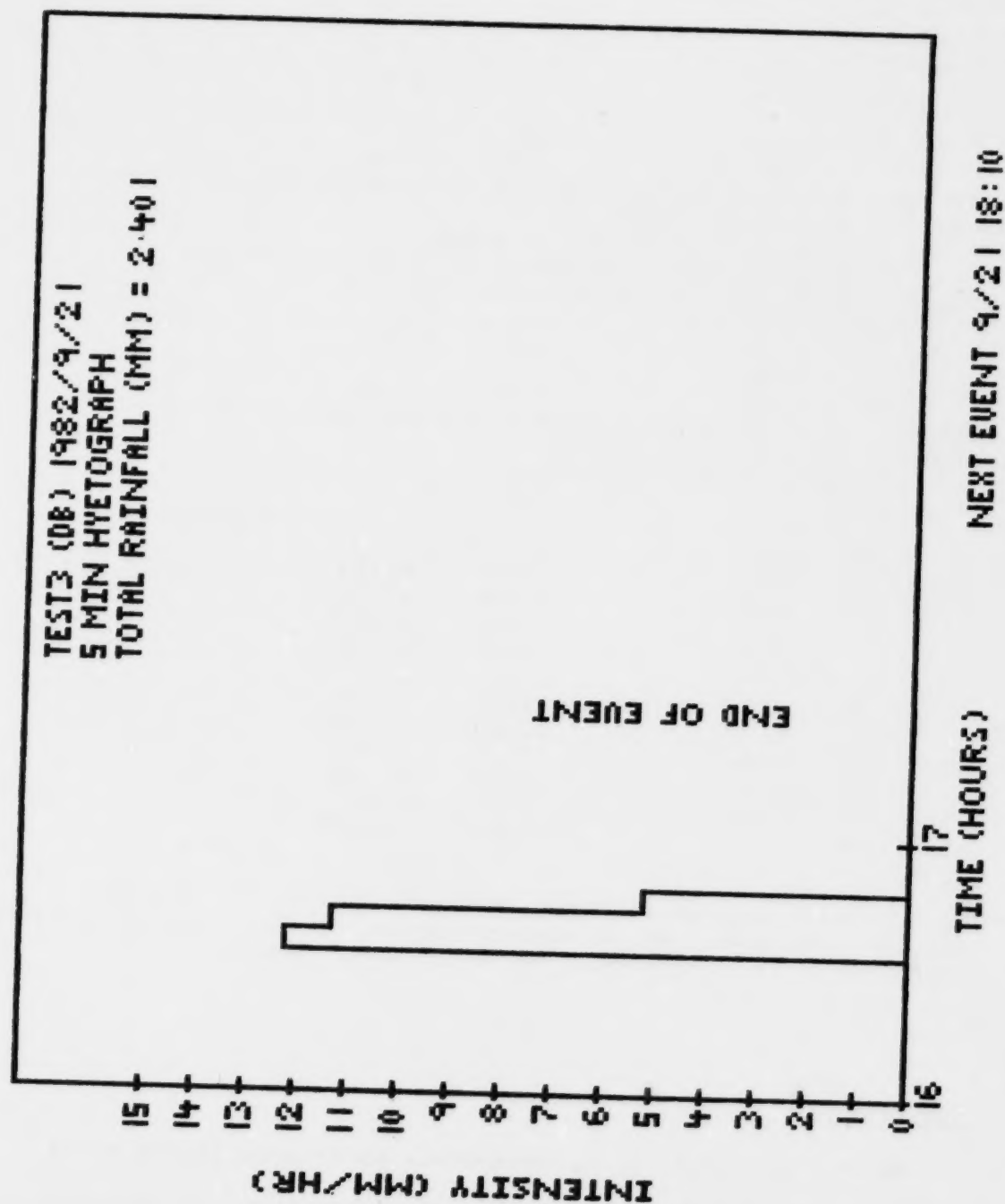
9	22	0:30	0	0		
9	22	4:30	1.14	68.4		
9	22	6:10	.152	9.12	1.14	1
9	22	6:15	.228	13.68		
WARNING - NON-ZERO RAINFALL HAS BEEN DETECTED FOR ZERO TIME						
CAUSE - DATA ENTRY ERROR						
TIME/RAIN PAIRS OUT OF PHASE						
HARDWARE ERROR DURING PROCESSING OF CASSETTE						
ACTION - DATA IGNORED PLEASE CHECK RESULTS						
9	22	6:25	.152	9.12		
9	22	6:30	.228	13.68		
9	22	6:40	.152	9.12		
9	22	6:45	.228	13.68		
WARNING - ZERO RAINFALL DETECTED WITH NON-ZERO TIME						
ACTION - DATA WILL BE IGNORED						
CHECK SOURCE DATA FILE						
9	22	6:55	.152	9.12		
					1.292	46
9	22	8:30	0	0		
9	22	8:33	.1444	8.664		
9	22	8:34	.2128	12.768		
9	22	8:35	.3496	20.976		
9	22	8:36	.1596	9.576		
9	22	8:37	.152	9.12		
9	22	8:38	.1596	9.576		
9	22	8:39	.3268	19.608		
9	22	8:40	.1292	7.752		
9	22	8:41	.0988	5.928		
9	22	8:42	.228	13.68		
9	22	8:43	.4408	26.448		
					2.4016	11
OBSERVED RAINFALL VOLUME FROM TOTALIZER =30 (MM)						
CALCULATED RAINFALL VOLUME FROM CASSETTE =8.86160001 (MM)						
DIFFERENCE BETWEEN OBSERVED AND CALCULATED VOLUMES =70.4613333 (%)						

1

CONCLUSIONS

Monitoring rainfall in the conventional manner using tipping bucket raingauges and strip chart recorders can be an imprecise, time consuming and expensive procedure. The use of a microcomputer-based data acquisition system

FIGURE 3



improves the accuracy and reliability of the data collected and reduces the cost per station. This in turn opens the way to increased coverage and/or increased network density.

Execution of simpler tasks such as data capture are ideally suited to peripheral personal microcomputers such as the APPLE II series and workalikes, for example the FRANKLIN ACE 1000. The low cost and simpler graphics results in cost effective data acquisition. The wide range of communication packages available permit data to be exchanged with a mainframe computer containing the central data base while still keeping data acquisition independent.

By transferring data capture tasks to a peripheral computer such as the FRANKLIN ACE from the larger multi-user PDP 11/23, thus freeing resources on the 11/23, the same quality and level of productivity can be maintained.

ACKNOWLEDGEMENTS

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REFERENCES

1. Haro, H., Kitai, R., and James, W., "Precipitation Instrumentation Package for Sampling of Rainfall", Institute of Electrical and Electronics Engineers (Transactions on Instrumentation and Measurement) Vol. IM32, No. 3, pp. 423-429, September 1983.
2. James, W., Haro, H., Robinson, M.A., and Kitai, R., "Hydrometeorologic Data Acquisition: Innovative, High Resolution, Programmable Instrumentation for Stormwater Management", Proceedings of the Conference on Stormwater and Water Quality Management Modelling. U.S. EPA, Washington, D.C., EPA 600/9-82-015 pp. 128-151, August 1982.

3. James, W., and Robinson, M.A., "Interactive Processors for Design Use of Large Program Packages", Canadian Journal of Civil Engineering, Vol. 9, No. 3, pp. 449-457. September 1982.
4. James, W. and Scheckenberger, R., "Storm Dynamics Model for Urban Runoff", International Symposium on Urban Hydrology, Hydraulics and Sediment Control, Lexington, Kentucky, pp. 11-18, July 25-28, 1983.
5. James, W. and Scheckenberger, R., "RAINPAC - A Program Package for Analysis of Rainfall Inputs in Computing Storm Dynamics", Proceedings of the Stormwater and Water Quality Modelling Meeting, U.S. EPA, Detroit, Michigan, April 12-13, 1984.
6. James, W. and Unal, A., "Evolving Data Processing Environment for Computational Hydraulics Systems", Canadian Journal of Civil Engineering, Vol. 11, No. 2 pp. June 1984.
7. Robinson, M.A. and James, W., "Rainfall Data Processing Manual for the Apple IIE Computer", CHI Publication No. R114, (about 70 pp.), January 1984.
8. Robinson, M.A. and James, W., "Continuous Variable Resolution Stormwater Modelling on a Microcomputer Using a Central Hydrologic Data Base Manager". Proceedings of the Canadian Hydrology Symposium, Quebec City, Quebec, June 10-12, 1984.

CHGTSM - A COMBINED HYDROLOGIC TIME SERIES AND TOPOGRAPHIC DATA BASE MANAGER

by

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ABSTRACT

The paper describes a boss microcomputer with hard disk connected to satellite microcomputers. Satellites support the independent portions of a very large water resources package while the boss system hosts the data base management system. The DBMS manages and distributes the time series data from the hard disk to the satellites via the boss whenever a request is made. The whole system is cost efficient since desktop microcomputers are used.

The paper describes the attributes and capabilities of the Computational Hydraulics Group Time Series Manager (CHGTSM). CHGTSM prepares, manages and distributes the CHG-Time Series Store. A communications, synchronization and security protocol to connect the satellites and the boss computer so that the time series store can be distributed for hydraulic applications will allow independent blocks of a large water resources package to work concurrently in independent but connected microcomputer systems.

INTRODUCTION

Hardware and software systems are evolving rapidly, but apart from program systems, little or no overall system building (including for example data acquisition and control) is evident in urban drainage applications. Continuous modelling requires use of climatological archive magnetic tapes for input and streamflow tapes as well as for calibration and validation purposes (Robinson and James, 1981). Large continuous modelling packages e.g. HSP-F and SWMM (Johanson et al., 1981; Huber et al., 1981) also include statistical processing and graphics output. The transition from event

modelling (the traditional design storm philosophy where startup values must be estimated) to continuous modelling now foreshadows problems of input and output data management (James, 1982). Special software is essential for I/O time series management, especially for local area networks. The network indicated comprises micro-computers radiating from a central disk operating system functioning on a boss CPU dedicated primarily to data base management (DBM), most of which is devoted to time series management (TSM). Each peripheral CPU is dedicated to separable blocks of logical routines, associated with, for example, precipitation analysis, runoff modelling, transport or drainage system networks, water quality modelling, cost-benefit analysis, or statistical post-processing. This can now be achieved at a fraction of single mainframe costs, and probably just as fast in terms of overall turnaround time (James and Robinson, 1981).

The background to our local area network was presented elsewhere (James and Unal, 1984). This paper describes in more detail the attributes and capabilities of the Computational Hydraulics Group Time Series Manager (CHGTSM). CHGTSM prepares, manages and distributes the CHG-Time Series Store (CHGTSS). In CHGTSM a group of users share a TS data base system in which the first input TS segment is processed for precipitation by one user and, when complete, the output TS is made available for that segment to the rainfall/runoff block of programs, perhaps under control of another user. A simple example is: precipitation TS is processed for year 1979 while rainfall/runoff TS is processed up to year 1978, the transport network is processed up to 1977, the sewage treatment plant processing up to year 1976, and the dispersion of the resulting pollutants in the receiving waters up to year 1975: all of this processing occurs concurrently.

CHGTSM was written to satisfy a portion of our group's requirements. It constitutes the TSM part of an overall DBMS which will be held in the boss computer of our group's computing system. Although derived from HSP-F, it will be seen that CHGTSM is basically a custom made, application oriented Time Series Manager with certain query attributes. It has yet to be expanded to include management routines for catchment-environment data, through geographic data banks.

In addition to the CHG-DBMS the boss computer will hold the communications, synchronization and a security protocol to avoid inconsistencies (i.e. to avoid concurrent access to the same piece of TS data by more than one satellite). The software to queue the incoming requests is called the Computational Hydraulics Group Distributed Processing Software (CHGDPS).

EVOLVING COMPUTER TECHNOLOGY

The growing number of independent computer systems used for scientific computing has created a need for local computer networks which should meet the following requirements: 1) relatively high data rates, 2) geographic distances spanning about one kilometer, 3) ability to support a sufficient number of independent devices, 4) simplicity, or the ability to provide the simplest possible mechanisms that have the required functionality and performance, 5) good error characteristics, good reliability and minimal dependence upon any centralized components or control, 6) efficient use of shared resources, particularly the communications network itself, 7) stability under high load, 8) fair access to the system by all devices, 9) easy installation of a small system, with graceful growth as the system evolves, 10) ease of reconfiguration and maintenance, and 11) low cost. In its simplest form, a network is a software mechanism for point to point transfer of files. First, though, the processors must be connected by a wire cable. This can be done by purchasing a null modem cable and swapping the transmit and receive lines of the two processors or by linking the processors with modems. The next step is to write the software for file transfer. This program turns the terminal of one processor into a virtual terminal for the other processor, and tests for a valid connection between processors (Birkenmeyer and Hopp, 1985).

An example of a current sophisticated local area network is Ethernet. The structure of which follows the ISO (International Standards Organization) OSI (open system interconnect) model, which divides a network functionally into seven layers.

Ethernet is a bus configuration where contention between multiple stations is resolved by a technique called carrier-sense multiple-access and collision detect (CSMA/CD). The transceiver provides the driver electronics for the cable, and the Ethernet interface unit provides address recognition, arbitration, and error detection. The Ethernet specification supports 10 M-bit/s performance for up to 100 nodes on a 500-meter segment of Ethernet coaxial cable, and each branch cable can be up to 50 meters long (Baha et al., 1984).

Local area networks give the user more power: the addition of a station expands not only accessibility but also speed and storage capacity (which is just opposite for mainframes). In a tightly coupled computer system, the processing units share memories and peripherals such as printers and disks. In a loosely coupled system, the processing units usually have their own memory and peripherals and communicate with each other over a communications network. A tightly coupled system is often called multi-processing, while a loosely coupled system is called distributed processing. In

both cases, different processing units perform different tasks and communicate with each other to coordinate their actions in a larger overall task. Each component operational module is written to coordinate its activities with other modules. Coordination of the segments of the program must recognize all possible conflicts, e.g. when two programs require to change the same data record at the same instant.

To prepare the distributed computing system of the 5th generation computers, users must develop valid procedural solutions for handling text and data. The system starts with the choice of database architecture; the priorities to be set in database integration, segmentation, distribution and maintenance; and the adoption of proper protocols. Substantial benefits are to be derived from a properly designed and implemented data base. "New standards and guidelines promulgated by professional groups must be observed in the design of computer based information systems. Unless our data assets are properly planned, in a few years it will be impossible to exercise effective control over them" (Chorafas, 1982).

DATA BASE MANAGEMENT SYSTEMS

As soon as several users require access to operational data which is being changed, and which is sufficiently large that personal copies of the data for each user would represent a significant load on the computer system, it is desirable to provide centralized control of the data bases. In this way, redundancy can be reduced, inconsistency can be avoided, data can be shared, standards can be enforced, security restrictions can be applied, integrity can be maintained, and conflicting requirements can be balanced (Date, 1982). A DBMS is a set of procedures and data structures that isolate the application from mundane details of storage retrieval, creation, modification and security of the data base - the user does not refer to physical storage locations but deals instead with the conceptual data structure. An individual DBMS user will generally be interested only in some portions of the total database and the user's view of that portion will generally be somewhat abstract when compared with the way in which data is physically stored. The external view is the content of the database as seen by some particular user. The conceptual view, on the other hand, is a representation of the entire information content of the database, again in a form somewhat abstract, in comparison with the way in which the data is physically stored. The component of the DBMS responsible for the external/physical conversion is called an access method.

Until recently microcomputer DBMS software could not replicate mainframe data structures. Mainframe DBMS's are increasingly adapting relational architectures in

which data structures are defined by a series of tables rather than by individual records, which now allows micro versions of sophisticated mainframe packages. Emerging integrated software links recently introduced by several software vendors permit microcomputers to tap directly into mainframe databases. Integrated software and local database approaches have merged providing microcomputer users with personal databases that are as easy to manipulate as spreadsheets.

These databases will be tied into intelligent networks that will transparently honor requests for data residing locally, on the mainframe, on another microcomputer, or on a database across the country (Freedman, 1984).

SCOPE OF CHG-TSM

Conventional, general purpose DBMS's, designed to organize information into a collection of different attributes with a common relationship, could be used for computational hydraulics applications. For example, the basic element of information in conventional DBMS used to maintain water resource data could consist of a data item such as stage, flow, or water temperature measurement for a particular station at a particular time. Construction of a time history of any of these variables under this scheme requires searching through several basic data items. An alternative is to use a block of sequential TS data as the basic element of information. This concept results in much more efficient access of TS data. The basic concept underlying CHGTSM is the organization of data into records of continuous, applications-related elements, as opposed to individually addressable data items. This general approach is more efficient for computational hydraulics applications than that of a general purpose DBMS because it avoids the processing and storage overhead required to assemble an equivalent record from a general purpose DBMS.

CHGTSM comprises nine independent programs:

- | | | |
|------------|-----------|-----------|
| 1) NEWTSS | 4) OPELBL | 7) RETRIV |
| 2) CHG-LAB | 5) QUERY | 8) UPDATE |
| 3) OPENFL | 6) INSERT | 9) PATCH |

1. NEWTSS

NEWTSS is part of HSP-F; it opens the TSS files and determines their dimension, by writing the TSS-description in the first record of each file. The TSS descriptor holds the number of records of the file and the number of words in each record. Its purpose is explained as follows: "NEWTSS is a stand-alone program which creates a Time Series

Store (TSS). It must, therefore, be run before a user can perform any runs which require data to be stored in, or retrieved from, the TSS. When running NEWTSS, the user specifies the size of the TSS, the record length, etc. NEWTSS can also be used to copy the contents of one TSS to another. This option is used if the existing TSS is too large or too small; the user creates a new TSS of the desired size and copies everything to it" (Johanson et al., 1980).

2. CHGLAB

CHGLAB is basically the Time Series Store Manager of HSP-F, but was modified to work in an 8-bit addressing mode and independent of the rest of the program. It writes into the dataset directory whenever a dataset is opened in a TSS file. It also writes the primary dataset label for each dataset which is opened. The general description of module TSSMGR is given in the HSP-F manual: "This module maintains a user's Time Series Store (TSS) and performs some housekeeping chores associated with the datasets in it. From the point of view of the computer's operating system, the TSS is a single file (which may be very large). However, many distinct time series can be stored in this file. This permits a user easily to keep track of the various time series with which he is dealing. Furthermore, he need only refer to a single disc file for all his time series input and output needs, no matter how many time series are involved. This simplifies communication with the computer system". (Johanson et al., 1980). HSP-F was originally developed on a HP-3000. We have made the necessary changes to work in an 8-bit mode. CHGTSM is presently also running on a PDP-11/23 under RXS-11M. This operating system provides an addressable memory of 32 K bytes to each of 8 users, but the compiler does not support formatted direct access, which is crucial for HSP-F. For the PDP-11/23 to function as the Boss processor of our local area network, two new programs were written, equivalent to NEWTSS and TSSMGR or CHGLAB, but using less memory space and unformatted direct access. The new programs are called OPENFL and OPELBL, and these take over portions of the responsibilities of NEWTSS and TSSMGR respectively.

3. OPENFL

OPENFL, written to replace NEWTSS of HSP-F, opens the TSS files required for the new TS information and is the first program to be run when preparing a database using CHGTSM software. OPENFL allows the user to open files with a minimum of 65 and a maximum of 185 real words. The user has to assign a file number for the TSS file to be opened. The dimensions (number of records and the number of words in a record)

can be assigned by the user or the program can be used to optimize these dimensions. All information is input through the terminal interactively. (Note that HSP-F reads a sequential disk file as input).

4. OPELBL

OPELBL was written to open dataset labels in a TSS file already opened by CHGTSM-OPENFL. The user assigns the dataset number to be opened in a TSS file, specifying the TSS file by calling its number.

5. QUERY

QUERY was written to access a database (TSS) originally prepared by HSP-F. (Note that HSP-F does not have a variable time step option.) QUERY retrieves the TS information from the TSS and prints into a user-requested external file using HSP-F access techniques. Addressing in the dataset is described as follows in HSP-F manual: "The REAL words in the dataset are logically treated as if they were numbered sequentially from unity starting with the first word of the first TSS record of the dataset. The label of the dataset always begins at the first word of the dataset. The key area of the label contains the address of the REAL word for the beginning of each calendar year stored in the dataset. The calendar years need not be stored in chronological order but the data within each calendar year is stored in chronological sequence. Access to TS information then takes place in two steps: the direct step to the TSS record containing the first word of the calendar year in question and the search to find the time interval within the year" (Johanson et al., 1980). Currently HSP-F can only move TS information from TSS to the core of the machine, to be processed by only HSP-F application routines. The TS information cannot be transferred to a disk file or printer. Thus HSP-F cannot at present be used as a DBMS for external applications.

6. INSERT

INSERT was written to prepare CHGTSM from scratch and comprises two subroutine groups which give the user two different options in preparing CHGTSS. The DCTSS group processes the TS information initially prepared with variable time steps whereas the UCTSS group processes the TS information with constant time step but allows the user to alter it while inserting data into CHGTSS. Both subroutine groups prompt the user for the TSS file number, dataset number and the year of the TS information to be activated. If the user is preparing the data files for the first time

and has collected data with variable time steps, he may use subroutine DCTSS. For this he has to prepare the data file with variable time steps. Certain control variables to structure the data base can either be supplied interactively or with the data file prior to execution.

If the data file is already prepared with constant time steps, the user will likely use subroutine group UCTSS. It provides an option to store the existing information with variable time steps through a process called compression. Subroutine group UCTSS analyzes the given constant time step TS data and informs the user of time spans where all the TS values are either zero or undefined. In a 185 real-word record size TSS file, the span could be three hours for one minute time step data, 6 hours for five minute time step data or 1 day for timesteps greater than five minutes. The user is asked if this span is to be stored in variable time steps, and the user supplies the new time step. Note that compression is possible only if a predetermined span of time has all values zero.

7. RETRIV

RETRIV was written to retrieve information from CHGTSS. Any span of time in a year may be retrieved (maximum 1 year, minimum 1 day). For each request, RETRIV asks if a new span is to be retrieved. Before writing the requested piece of TS to an external file, RETRIV will ask if aggregation/disaggregation is necessary. If the answer is NO, the TSS information will be retrieved the way it was stored in the database. If the answer is YES and the output file is to be prepared with a constant time step, RETRIV will aggregate/disaggregate all the data to the user supplied constant time step. If the output file incorporates variable time steps then RETRIV will inform the user about the contents of each logical record in the database (i.e. for time steps greater than five minutes it will give month of the year, the day and the time step of the day; for time steps smaller than or equal to five minutes, it will give the month, the day and the time span of the day - 3rd 1 hour time span, 5th 3 hour time span etc. - and the time step). In addition RETRIV will warn the user if the content of a logical record is all zeros (or undefined) and request the new time step required in the output data-file. RETRIV will decide to aggregate or disaggregate by comparing the timestep given by the user and that of the stored TS information. Any logically existing records can be aggregated/disaggregated by the user. Note that certain logical records may not exist in the physical sense. If a physically non-existing record is to be retrieved with no aggregation/disaggregation specified, 60 minute time intervals are used as default.

8. UPDATE

UPDATE updates the information in CHGTSS, such as the contents of a dataset of previously inserted TS information. The user can access any span of time within each year for existing information. The minimum accessible time span is one day. UPDATE informs the user about the status of the information in each record. Either the control variables or the TS information can be subject to an update in each logical record. Any logical record which does not correspond to a physical record cannot be updated.

9. PATCH

PATCH was written to save portions of TS data which do not necessarily start at the beginning of a year and end at the end of the year. Since this condition can occur during a distributed data processing application, this software is designed to work with minimum user intervention. PATCH will process TS information initially prepared with variable time steps. The control variables to structure the dataset and TS information are read from a sequential disk file. On the other hand, the TSS file, dataset number and the year of the requested TSS information are input interactively. When an application routine supplies the input file to CHGTSM, for a time space shorter than a year, PATCH ensures that the periods before and after that time span are filled with zeroes or undefined values. Note that an output file from an application package acts as an input file for CHGTSM and vice-versa.

STRUCTURE OF CHGTSS

CHGTSS comprises several TSS files, each identified by a unique name, (e.g. CHGTSS1, CHGTSS2, etc.) Files are composed of a variable number of records, each file having a record size in words assigned by the user.

Although all the records in a file have the same physical size the user sees variable size records. The maximum possible number of words in a record is a function of the system in use. CHGTSM optimizes the usage of disk space when the record size assigned is 65 real words. When TSS files are prepared the user has an option to assign higher record sizes. These options are 125 real words and 185 real words. Any record sizes greater than 185 real words will function as 185 real words. Since CHGTSM can work with 8-bit machines care must be taken if record sizes are to be increased. The first record of each TSS file is called the TSS descriptor, defining the characteristics of the TSS. Certain of these characteristics will vary from one TSS to another and it is therefore necessary that the TSS be self descriptive.

As in HSP-F, the first record of each TSS file, called the TSS Descriptor, defines the characteristics of the TSS. The TSS Descriptor is followed by the TSS Directory, which describes itself and the time series datasets in the TSS.

The number of TSS records to allocate to the directory is set by the user when the file for the TSS is first created. This first estimate may need to be changed as the TSS is used. Thus, for the purposes of dataset manipulation, the TSS directory is not distinguished from a time series dataset. (Johanson et al., 1980).

TSS directory is followed by the first dataset label in the file. This label occupies 2 records. The first is called the primary dataset label. The second is called the secondary dataset label. PDSL contains information describing the contents of the dataset. SDSL serves as an index to the first day of each month of each year in the dataset. Thus CHGTSM has direct access to the first day of each month of any year. Between the first day of each month the search is done sequentially.

The TSS dataset labels are followed by the actual time series information. Each dataset may contain several years of time series information all belonging to a unique source. TS information can be stored with variable time steps for each day of the year. As an example, in a TSS file with 185 real-words records, if the dominant time step exceeds 5 mins. for a particular day, that day occupies one record. If the dominant time step is equal to 5 MINS each 6 HR span occupies one record ($\therefore$ 4 logical records to cover the day). If the dominant time step is equal to 1 min each 3 hr span occupies one record ($\therefore$ 8 logical records to cover the day).

The first 5 words in each record are called control variables. Control variables are DAY (day of the month), DOMDEL (dominant time step), COMFLG (compression flag), COMDEL (compression time step), NTFRAM (number of time frames in that record). COMDEL is always equal to DOMDEL if DOMDEL exceeds 5 mins. If DOMDEL is equal to 5 mins. (or 1 min), COMDEL can be greater than or equal to 5 mins (or 1 min). If DOMDEL is not equal to COMDEL at any time span of a given day COMFLG is equal to 1 which means compression exists. In this case NTFRAM is determined by COMDEL.

In each record the control variables are followed by the actual time series data. The number of TS values in any record is determined by NTFRAM.

It is clear from foregoing discussion that CGH-TSM not only stores the TS with variable time steps with respect to days but also in certain cases within the days. Those cases are when the dominant time step is 5 min or 1 min. 1 min and 5 min time steps are very important for rainfall data acquisition since our raingages supply the non-zero rainfall data with these time steps. When this data is to be inserted into the

database, the database administrator (DBA) has to choose his strategy. For example if the rainfall is not intense and the data spans relatively short intervals the DBA may decide to structure the TSS file with 65 real-word records. This will give a finer resolution and save significant amounts of disk space. If the rainfall is intense and continuous for relatively long periods of time, the DBA should structure the TSS file with longer record sizes because shorter record size will not make any difference to the physical amount of disk space used. On the other hand both the insertion and retrieval procedures will be much faster for TSS files with longer record sizes.

It is important to understand why fewer resolutions (i.e. 65 real-words) saves a significant amount of disk-space compared to a coarser resolution (i.e. 185 real-words). If the data base is structured with 65 real-words, a 1 min dominant time step day will be composed of 24 logical records, each 1 hr length. But the number of physical records to represent that day may be less than 24. Because CHG-TSM does not store the compressed time series records with COMDEL equal to 60 min. The DBA has an option to assign 60 to COMDEL whenever he finds a 1 hour time span with all zero values. Infact this is done by default when using PATCH to structure the data base. Although this record is not stored (physically) in the data base its existence is acknowledged by the access method. The user retrieving information from the TSS finds the physically non-existing record displayed in his logical view.

If the record size is 185 real words in a given TSS file the DBA has to find a 3 hr time span with all zero values so that he can compress with COMDEL equal to 60 mins. This will be more difficult to find compared to a 1 hr time span with all zero values.

Once a TS record is compressed with COMDEL = 60 mins the user can retrieve it just like physically existing records. For example a user can aggregate or disaggregate a physically non existing record to any time step.

GEOGRAPHIC DATA BASES

A geographic data base, a stored computer file of map data, can be created in two ways: using grid cell or x, y coordinate is divided into a grid cell method. In the grid cell method the study area is uniform grid and all data encountered within averaged. The x,y coordinate methods use point coordinates to define the points, lines and polygons describing geographic variation.

Grid cell data banks have the following advantages (HEC, 1978):

1. The grid cell provides an easy way to collect data. In the simplest method, one lays a plastic overlay of grid cells on top of maps or aerial photos and interprets information directly.

2. Computer storage and the subsequent access and processing of this matrix of information is extremely straightforward.
3. Editing of stored data is also straightforward.
4. Due to its consistent nature, the grid cell can be used for both discrete and continuous types of data analysis. Simple computer programs allow the user to aggregate many basic data items into the grid cell for different types of display.
5. Data encoded by x,y coordinate methods can be directly translated to a grid system by available programs.
6. Point data of a continuous surface nature (e.g. air pollution records) can be counted to a grid cell scheme by interpolation.

The major disadvantage is the loss of resolution. Resolution and cost increases as the size of grid cells decreases. choice of the grid cell size and shape is critical, different data banks can be prepared using different cell sizes, but any given data bank will contain only one grid cell size and shape.

Recently the US Army Corps of Engineers Hydrologic Engineering Center has been developing computer programs in the area of catchment-environmental DBM as well as TSM (called HEC-SAM, HEC-DSS, based on Grid Cell Data Banks). The HEC-SAM system is more than a general purpose spatial data procedure. It has the capability to assess hydrologic, flood damage, and environmental consequences of development, perform wildlife habitat evaluations such as the U.S. Fish and Wildlife Habitat Evaluation Procedure, perform boolean and overlay analysis, and produce a variety of computer graphics. "The general analytical strategy that comprises HEC-SAM is to: a) assemble and catalog basic geographic and resource information into a computer data bank, b) forecast and place into the data bank selected alternative future development patterns, formulate an array of management alternatives, c) perform comprehensive assessments of the development scenarios of interest, and d) recycle for additional alternatives" (Davis, 1981). The following analyses may be carried out (HEC, 1978):

1. Hydrologic Analysis

- determination of subbasin area statistics
- determination of runoff coefficients
- computation of subbasin precipitation
- spatial display of erosion, sediment deposition and dredge material sinks

2. Economic Analysis
 - determination of land use statistics
 - computation of damage to development in flood plain
 - formulation of nonstructural plans
 - display of assessed values
 - determination of proximity to transportation corridor
3. Environmental Analysis
 - determination of vegetative cover
 - determination of wildlife habitat zones
 - computation of land surface erosion parameters
 - computation of urban storm runoff quality parameters
 - determination of locational attractiveness indexes
 - assessment of general geographic impact
 - coincident tabulation of habitat reduction for use in scenarios of potential impacts
4. Social Analysis
 - display spatial distribution of income level
 - display ethnic zones
 - assess age variations
 - compute census statistics

DISTRIBUTED PROCESSING OF CENTRALIZED DATA

Where a user group requires a variety of computer hardware and/or operating systems software, it becomes desirable to develop a distributed data processing system (i.e. local area networks). Large hydraulic engineering application programs that enjoy a wide distribution or utilization, are the first to be adapted to a distributed processing system, because these systems offer reduced computational costs as well as maximum utilization of manpower. Many problems in computational hydrology are equally if not more demanding of file access than of CPU time. This is especially true when TSS manipulations are involved. In this case a smaller, less expensive computer system (i.e. smaller word sizes, slower CPU cycles, and less sophisticated instructions) with adequate file use is often just as effective a solution but at a significantly lower cost. Thus an intelligent disk storage system configured as a stand alone DBMS accessible by several microcomputers may represent a more sophisticated solution. The intelligent, expandable, disk controller handles all data access including indexing, searching,

buffering, deblocking and storage management functions. Such a system makes more efficient use of each processing unit, so that the user obtains better response times.

The computing system we propose is a boss computer connected to a hard disk at one end and to a satellite computer system at the other end. Satellites support the independent portions of a very large water resources package while the boss system hosts the DBMS. The DBMS manages and distributes the TS data from the hard disk to the satellites via the boss whenever a request is made. A typical request will occur when an application module (satellite) finishes processing TS data and attempts to save the TS in CHGTSS (Disk) or needs more data to process from CHGTSS.

Basically given a computer system, a program has two resources: time and memory space. The space requirement is the amount of memory and peripheral storage used by a program. That means the space requirement can be broken down into the amount of primary and secondary storage that is needed.

Program space refers to the actual space in memory that contains the program. It is very dependent upon the hardware/software configuration of each computing system. Instruction set, addressing methods, operating system conventions all play a role in the size of a program. There are very few principals that can be developed regarding the measurement of the space requirements of a program.

As far as data space is concerned, there are some general observations that can be made about the amount of data space required by a program and how the size of the data space depends upon the algorithms used and program input. Techniques that are used to estimate data storage requirements are similar to the techniques that are used to measure time requirements.

Estimating the amount of time used by the program can be approached in several different ways (i.e., CPU time, WALL time, I/O time). The amount of time and space required by a program directly depends on: 1) the actual hardware, 2) the algorithmic structure of the program, and 3) the data processed by the program.

During a distributed data processing application it is important to synchronize the processing in each individual node of the local area network. It will never be possible to optimize the computing such that each node is actively computing all the time instead of waiting for data to arrive. On the other hand of the nodes end up waiting for data for an unreasonable span of time a single processor can do the whole job with comparable efficiency.

Consequently its important to know how much time a node will take to finish computing once the data is downloaded from the DBMS. Program measurement techniques make it possible to obtain timing as a function of basic execution time of a

sequence of instructions and the amount of constraint data. By constraint data we mean the data which controls the number of executions of any loop in the algorithm.

Timing functions of routines are used to inform the user about the expected execution time in a given processor for a specified amount of data (or vice versa) prior to or during execution.

CONCLUSIONS

The adoption of 16-bit and 32-bit personal computers, with their improved computational power, and reduced costs, inevitably leads to continuous modelling. This turns up problems of management of large time series. As software costs become a significant part of the total computer costs, data base management systems will ultimately be required by every group of engineers sharing a computational hydrology data base. Data base software significantly reduces the cost of software development in several ways: programmers are not required to maintain their own copy of the data base and data file, an internal directory describing the relationship among the data is automatically maintained and is available to all users, and programmers are not required to write special purpose programs to fit the data (Gagle and Koehler, 1981).

The novelty in our approach lies in getting away from a single processing unit with a single huge memory, performing in a sequential step-by-step path. This system is replaced by a number of simple processing elements, each endowed with its own small memory. The processors themselves are arranged in a hierarchy. At the top of the hierarchy, lies the boss processor which holds CHGTSM and CHGDPS, to access the database. Satellites hold the application packages and their share of the CHG-DPS.

We are developing integrated computational hydraulics systems by interfacing distributed data processing utility modules with the regular operational or application modules required and/or developed by other members of our group.

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REFERENCES

Baha, J.J., Weurel, H.M., Willits, J.L. A Local Area Network for the HP 1000 Series 500 Computers. HP Journal, March 1984, Volume 35, Number 3, pp. 22-28.

- Birkenmeyer, D.R., Hopp, R.J., (1984) The RSX Multi-Tasker, Decus, February, Section 1, pp. 48 to 50.
- Chorafas, D.N. (1982). Databases for Networks and Minicomputers. Petrocelle Books, Inc. 281 pp.
- Date, C.J. (1981). An Introduction to Database Systems. 3rd Ed., Addison-Wesley Publishing Co., Massachusetts, 574 pp.
- Davis, D.W. (1981). Data Management Systems for Water Resources Planning. Technical Paper No. 81. Hydrologic Engineering Center. U.S. Army Corps of Engineers, 11 pp.
- Freedman, H.D. (1984). Tapping the Corporate Database. High Technology, April, 1984 pp. 26 to 28.
- Gagle, M. and Koehler, G.J. (1981). Data-base Management Systems: Powerful Newcomers to Microcomputers. Byte. p. 97-122.
- HEC (1978). Guide Manual for the Creation of Grid Cell Data Banks. HEC, U.S. Army Corps of Engineers, September, 1978, 74 pp.
- Huber, W.C., Heaney, J.P., Nix, S.J., Dickinson, R.E. and Polmann, D.J. (1981). Stormwater Management Model, Users Manual, Version III. Municipal Environmental Research Center, U.S. Environmental Protection Agency, Cincinnati, 300 pp.
- James, W. (1982). Continuous Models Essential for Detention Design. Conference on Stormwater Detention Facilities Planning Design Operation and Maintenance, Co-Sponsored by the Engineering Foundation and the Urban Water Resources Research Council of the A.S.C.E. Henniker, New Hampshire, pp. 163-175.
- James, W. and Robinson, M.A. (1981). Coordinated Multi-Processing for Large Scale Data Acquisition and Simulation of Urban Drainage for the Hamilton Wentworth Region. 5th Canadian Hydrotechnical Conference. University of New Brunswick, Fredericton, N.B., pp. 801-815.
- James, W. and Unal, A. (1984). Evolving Data Processing Environment for Computational Hydraulics Systems. Canadian Journal of Civil Engineering, Vol. 11, No. 2.
- Johanson, R.C., Imhoff, J.C., Davis, H.H. (1981). Users Manual for Hydrological Simulation Program - FORTRAN (HSP-F) Release 7. U.S. EPA, Athens Georgia.
- Robinson, M.A. and James, W. (1981). Continuous SWMM Quality Modelling for the City of Hamilton using Atmospheric Environment Service Data. Proceedings of the Conference on Water Quality and Stormwater Management Modelling, Niagara Falls, Ontario, US EPA, pp. 469-492.

MODIFICATIONS TO THE QUAL-2 (SEMCOG)
WATER QUALITY MODEL

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I. Introduction

This paper will outline and briefly discuss recent changes made by NCASI to the water quality model QUAL-2 (SEMCOG version). These code changes resulted from the fact that QUAL-2 has seen extensive use in recent years as a tool in water resource planning and waste load allocation studies. Furthermore, many modifications (both major and minor) have been made to the original code to reflect differing assumptions of algal growth, nutrient, and light interactions. In addition, a number of computer code errors were uncovered following extensive model testing (1). Consequently, the "QUAL-2" acronym no longer denotes the same model to all users, because substantial differences exist in the several versions reported in the public domain. A need, therefore, existed to: (a) combine significant state-of-the-art advances in algal kinetics into one centrally available model, (b) provide a thorough documentation and testing of the synthesized code, and (c) provide guidance for its use. This paper will address the first of these needs. The new model will have the acronym "QUAL-2E" to represent the enhanced model capability. Partial financial support for the work is acknowledged from the EPA Center for Water Quality Modeling (CWQM), Athens, Georgia.

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The paper will first discuss the historical development of the QUAL-2 model and summarize some of the differences between those versions which exist in the public domain. Specific computer code changes implemented by NCASI will then be schematically presented and briefly discussed.

II Historical Development of QUAL-2 Models

As discussed in NCASI Technical Bulletin No. 391 (1), the version of the model QUAL-2 documented and tested by NCASI was that obtained from the Southeast Michigan Council of Governments (SEMCOG) with assistance from EPA Region V. The model was originally developed by Water Resources Engineers, Inc. (now Camp, Dresser, McKee, Inc.) and is fully described in two reports (2,3). This version was originally selected by NCASI because of the availability of quality documentation and user manuals.

This version of QUAL-2 has seen extensive use and application by both consultant and regulatory personnel in recent years. More importantly, the model is one of several maintained and supported by the EPA Center for Water Quality Modeling in Athens, Georgia. The EPA has sponsored several workshops throughout the U.S. to provide training and guidance for the use of QUAL-2. NCASI has participated in these workshops and has provided EPA with copies of Technical Bulletin No. 391 (1) which presents the updated computer code and documentation manual. In addition, NCASI staff has assisted numerous member companies and their consultants in implementing the QUAL-2 model code on differing computer systems.

Several other versions of the QUAL-2 model have been uncovered in NCASI reviews of model applications. The general input/output, hydraulic and water quality capabilities, and other special features are compared in Table 1. The five separate models compared are QUAL-2 versions attributed to SEMCOG, the Texas Water Development Board (TWDB) (4), Meta Systems, Inc. (5), Upper Mississippi River 208 (6), and the Wisconsin Department of Natural Resources (7). Major differences between models are noted under the nitrogen and algae simulation routines.

NCASI's application of the SEMCOG version of QUAL-2 on the Ouachita River Basin provided the justification for the current re-examination of the QUAL-2 model code (8). In this study, water quality simulations were examined for portions of a deep, low velocity river system where algal dynamics were dominant. In this application algal cells

were productive in a small zone near the surface where they could expect to grow at their optimum rate. The routine in QUAL-2 requires that the algae be productive over the entire water column under some artificially low light level. The light level calculated is that single average value necessary to achieve the same algal productivity that actually occurs over a 24 hour diurnal cycle. As the river became deeper near the impoundment, the model distributes the available light over a larger volume and produces both lower growth rates and D.O. levels. The field data, however, showed that more algal productivity was required in these areas to simulate high observed D.O. levels.

An additional problem in the Ouachita River study was that of high sensitivity of simulated D.O. to the algal parameters. Calibration/verification was difficult to achieve in some cases for the variables ammonia, nitrate, algal biomass concentration, and D.O. For example, D.O. and algal biomass concentrations could be calibrated with reasonable parameter values (i.e., within range of published values), but simulated nitrogen species concentrations would then be significantly different than measured values. The subsequent investigation study of algal cycle modeling state-of-the-art served as the basis for the QUAL-2 update reported in this summary.

III NCASI Modifications to the QUAL-2 (SEMOG) Code

The following tentative list summarizes the modifications made to the QUAL-2 (SEMOG) code by NCASI.

(1) Algal, Nitrogen, Phosphorus, Dissolved Oxygen Interactions (Figure 2)

- organic nitrogen
- organic phosphorus

(2) Algal Growth Rate (Figures 3, 4, 5, and 6) .

- growth rate dependent upon both NH_3 and NO_3 concentrations
- preference factor input for NH_3
- algal self-shading
- three light functions for growth rate attenuation
- three growth rate attenuation options
- four diurnal averaging options for light

(3) Temperature (Figure 7)

- link to algal growth via solar radiation
- default temperature correction (θ) factors

(4) Dissolved Oxygen (Figure 8)

- new standard methods D.O. saturation function
- traditional SOD units ($\text{gm/m}^2 \text{ day}$)
- dam reaeration option

(5) Arbitrary Non-Conservative

- first order decay
- removal (settling) term

(6) Hydraulics

- input factor for longitudinal dispersion
- test for negative flow (i.e. withdrawal greater than flow)

(7) Input/Output Modifications (Figure 9)

- new coding forms
- local climatological data echo printed
- enhanced steady state convergence summary
- five part final summary including plot of D.O. and BOD

These algal, nitrogen, phosphorus and dissolved oxygen interactions in this listing are shown schematically in Figure 1. For the sake of comparison, the SEMCOG interactions for these same constituents are also displayed.

IV Summary

The computer code for this program will be completed during the spring of 1984 and submitted to the EPA CWQM. Subsequent updating of the documentation manual (NCASI Technical Bulletin No. 391) will follow in late 1984. It is important to note that the resulting program will require thorough and detailed documentation to minimize misuse by inexperienced users. Several input parameter values have multiple meaning, depending upon the algal, nutrient, and light options chosen. The enhanced output format, on the other hand, will provide greater detail than other versions of QUAL-2. This detail includes several diagnostics expected to be helpful in simulating and dis-

solved oxygen concentrations in river systems dominated by algal productivity.

V Literature References

- (1) "A Review of the Mathematical Water Quality Model Qual-2 and Guidance for its Use", NCASI Technical Bulletin No. 338, October 1980 (Revised No. 391, December, 1982).
- (2) "Computer Program Documentation for the Stream Quality Model Qual-II", prepared for Southeast Michigan Council of Governments, Detroit, Michigan (July, 1977).
- (3) "Users Manual for the Stream Quality Model Qual-II", prepared for Southeast Michigan Council of Governments, Detroit, Michigan (July, 1977).
- (4) "Qual-TX User's Manual (Draft)", Texas Water Development Board, Austin, Texas (June, 1981).
- (5) "Calibration and Application of Qual-2 to the Lower Winooski River: Preliminary Studies", prepared for state of Vermont by Meta Systems, Inc. (July, 1979).
- (6) Norton, W.R., et al, "Computer Program Documentation for the Stream Quality Model-Qual-II", for EPA Contract 68-01-1869 (August, 1974).
- (7) Patterson, D., et al, "Water Pollution Investigation Lower Green Bay and Lower Fox River", Wisconsin DNR, (EPA 68-01-1572), (June, 1975).
- (8) "A Study of the Selection, Calibration, and Verification of Mathematical Water Quality Models", NCASI Technical Bulletin No. 367, (March 1982).

TABLE 1 COMPARISON OF QUAL - 2 MODELS

Model Capabilities	SEPCOG		TWB		Model Version		Mod. Upper Miss. 208
	Steady State, Quasi-dynamic	Steady State	Steady State	Steady State, Revised Quasi-dynamic, Diel curve	Steady State, Revised Quasi-dynamic, Diel curve	Steady State, Revised Quasi-dynamic, Diel curve	
Temporal Simulation							Wisconsin DNR
CBOD	Steady state, BOD-5, BOD-U Settling	Same as SEPCOG	Same as SEPCOG	Bottle rate to stream rate conversion	Same as SEPCOG	Same as SEPCOG	Same as SEPCOG
SOD	Zero order	Zero order (revised units)	Zero order (revised units)	Same as SEPCOG	Same as SEPCOG	Same as SEPCOG	Dual first order, Bottle rate to stream rate conversion
Reseration	8 options	9 options, $k_2 > 2t/D$	9 options, $k_2 > 2t/D$	7 options, dam reseration	7 options, dam reseration	7 options, dam reseration	8 options
Nitrogen	NH3, NO2, NO3, algae, benthic source/sink	Organic-N, NH3 (NO2+NO3), algae, benthic source/sink, macrophytes, denit., 02 dependent rates	Organic-N, NH3 (NO2+NO3), algae, benthic source/sink, macrophytes, denit., 02 dependent rates	Organic-N, NH3, NO2, NO3, algae, benthic source/sink	Organic-N, NH3, NO2, NO3, algae, benthic source/sink	Organic-N, NH3, NO2, NO3, algae, benthic source/sink	Organic-N, NH3, NO2, NO3, W gas, algae, benthic source/sink, denit., 02 dependent rates
Phosphorus	Dia-P, algae, benthic source/sink	Dia-P, algae, benthic source/sink, macrophytes	Dia-P, algae, benthic source/sink, macrophytes	Dia-P, total-P, algae, benthic source/sink	Dia-P, total-P, algae, benthic source/sink	Dia-P, total-P, algae, benthic source/sink	Same as SEPCOG
Algae	Growth cycle (chla); NO3, P, light (I) benthic sink	Growth cycle (chla); NH3, (NO2+NO3), P, light (I), benthic sink, self-shading, photoinhib., growth cycle (macrophyte); as above	Growth cycle (chla); NH3, (NO2+NO3), P, light (I), benthic sink, self-shading, photoinhib., growth cycle (macrophyte); as above	Growth cycle (chla); NH3, NO2, P, I, benthic sink, self shading, P-N, diel curve	Growth cycle (chla); NH3, NO2, P, I, benthic sink, self shading, P-N, diel curve	Growth cycle (chla); NH3, NO2, P, I, benthic sink, self shading, P-N, diel curve	Growth cycle (chla); NH3, NO2, P, I, benthic sink, self shading, photoinhib.
Coliform	First order	Same as SEPCOG	Same as SEPCOG	Same as SEPCOG	Same as SEPCOG	Same as SEPCOG	Single constituent, conservative or first order
Non-conservative	First order	First order, settling	First order, settling	Same as SEPCOG	Same as SEPCOG	Same as SEPCOG	See above
Conservative	Three	Two	Two	Same as SEPCOG	Same as SEPCOG	Same as SEPCOG	See above
Temperature	Steady state heat balance, dynamic heat balance, fixed rate corrections	Steady state heat balance, user specified rate corrections	Steady state heat balance, user specified rate corrections	Steady state heat balance, (dynamic heat balance?), fixed rate corrections	Steady state heat balance, (dynamic heat balance?), fixed rate corrections	Steady state heat balance, (dynamic heat balance?), fixed rate corrections	See above, fixed rate corrections
Hydraulics	Rectangular channels, trapezoidal channels, flow augmentation	Rectangular channels, tidal downstream boundary, flow augmentation	Rectangular channels, tidal downstream boundary, flow augmentation	Rectangular channels, travel time, flow augmentation	Rectangular channels, travel time, flow augmentation	Rectangular channels, travel time, flow augmentation	Rectangular channels, flow augmentation
Loads	Point, distributed, headwater, tributary	Multi-point at one element, distributed, headwater, tributary	Multi-point at one element, distributed, headwater, tributary	Same as SEPCOG	Same as SEPCOG	Same as SEPCOG	"dynamic" point, "dynamic" headwater, distributed, tributary
Input/Output	Standard QUAL I/O	Reorganized, line printer plot	Reorganized, line printer plot	Standard QUAL I/O (line printer plot?)	Standard QUAL I/O (line printer plot?)	Standard QUAL I/O (line printer plot?)	Reorganized, VTY output option, line printer plot, Calcomp plot
Special		Sensitivity analysis	Sensitivity analysis				Suspended solids simulation

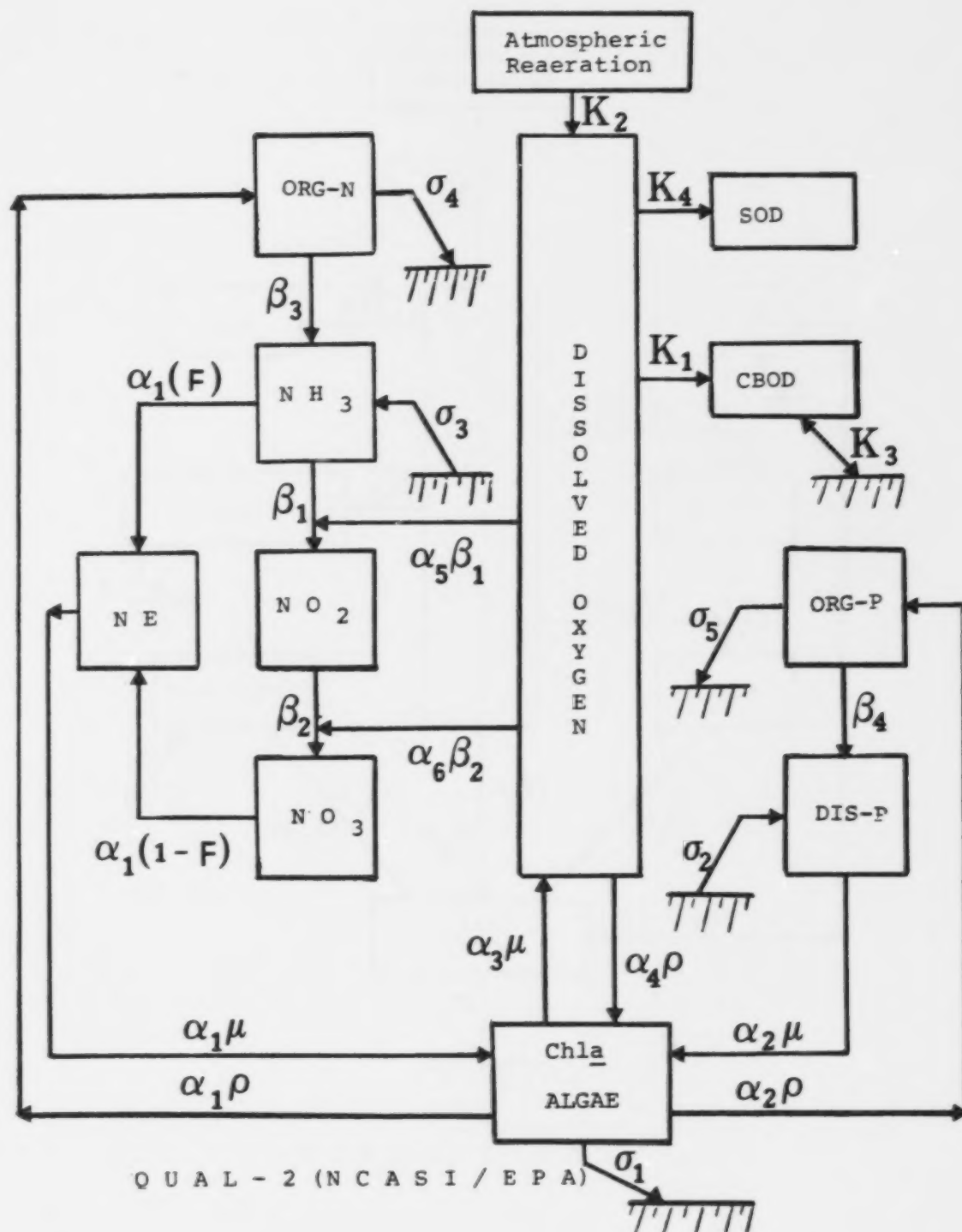
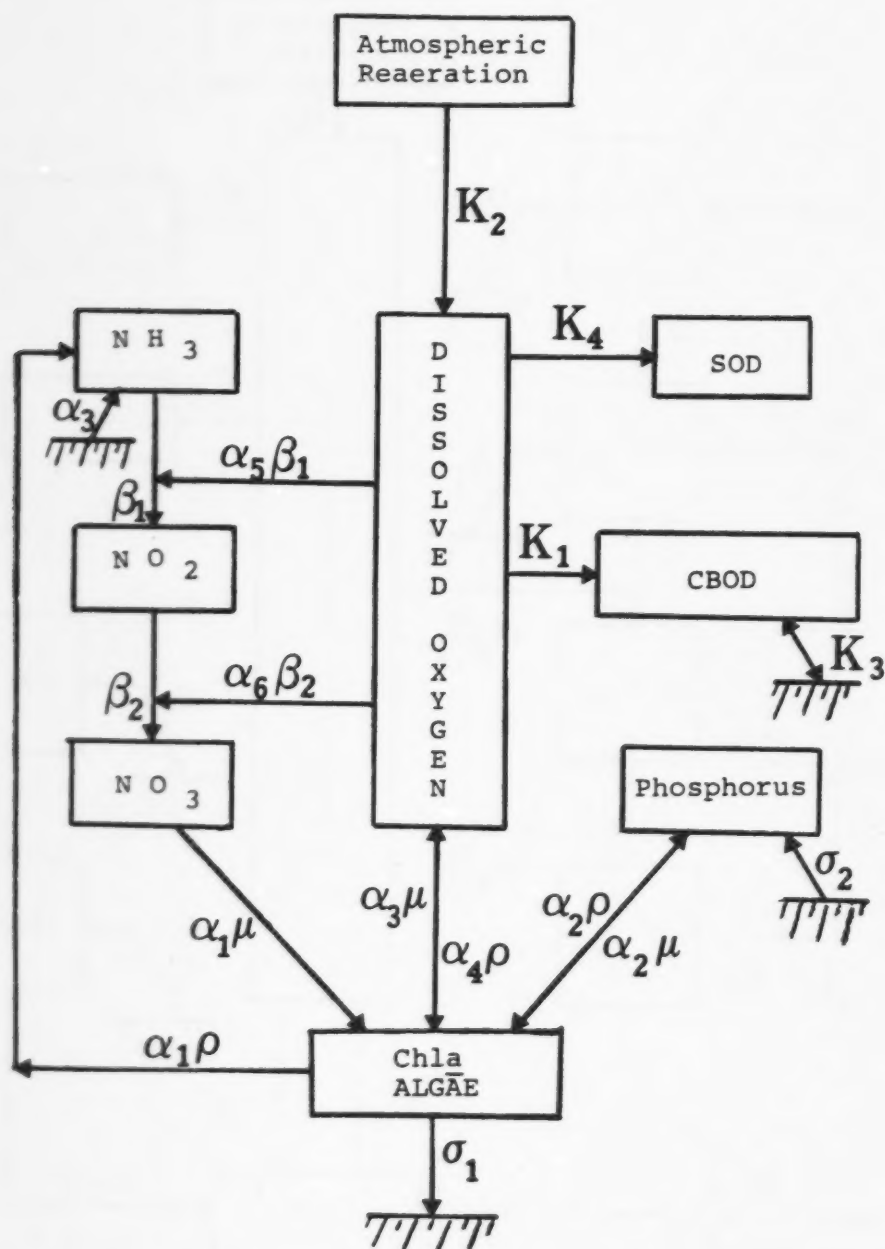


FIGURE 1



Q U A L - 2 (S E M C O G)

FIGURE 1 (CONTINUED)

FIGURE 2
ALGAL, NITROGEN, PHOSPHORUS, OXYGEN INTERACTIONS

Nitrification Inhibition at Low DO

* Correction Factor (QUAL-TEXAS)

$$\text{CORDO} = \frac{1.2 * \text{DO}}{1.56 + \text{DO}}$$

* Applied to ammonia and nitrite oxidation rates.

Ammonia: $(\beta_1)_{\text{inhib.}} = (\beta_1)_{\text{input}} * \text{CORDO}$

Nitrite: $(\beta_2)_{\text{inhib.}} = (\beta_2)_{\text{input}} * \text{CORDO}$

* Magnitude of correction factor.

<u>DO (mg/l)</u>	<u>CORDO</u>
0	0.00
1	0.47
2	0.67
3	0.79
4	0.86
5	0.91
6	0.95
7	0.98
8	1.00

FIGURE 3
ALGAL GROWTH RATE

Nutrient Attenuation Factors

* Nitrogen: $NE = NH_3 + NO_3$

$$FN = NE / (KN + NE)$$

where:

FN = Nitrogen attenuation factor
KN = Nitrogen half saturation
coefficient

* Phosphorus:

$$FP = PO_4 / (KP + PO_4)$$

where:

FP = Phosphorus attenuation factor
KP = Phosphorus half saturation
coefficient

Algal Preference for Ammonia

$$f = \frac{(P_1) (NH_3)}{(P_1)(NH_3) + (1-P_1)NO_3}$$

where:

P_1 = Algal preference for ammonia nitrogen (0-1.0)
0 = NO_3 1.0 = NH_3

f = Fraction of algal nitrogen uptake from
ammonia pool.

FIGURE 4
ALGAL GROWTH RATE

Algal Self-shading

* General Equation

$$\lambda = \lambda_0 + \lambda_1 A + \lambda_2 A^{2/3}$$

where:

λ = light extinction coefficient

λ_0 = non-algal extinction

λ_1 = linear algal extinction coefficient

λ_2 = non-linear algal extinction coefficient

A = algal concentration ($\mu\text{g Chla/l}$)

* Special Cases

- No Self-shading (SEMCOG)

$$\lambda_1 = \lambda_2 = 0$$

- Linear Self-shading (META)

$$\lambda_1 \neq 0 \quad \lambda_2 = 0$$

-Non-linear Self-shading (TetraTech)

$$\lambda_1 \neq \lambda_2 \neq 0$$

e.g. $\lambda = \lambda_0 + 0.0088A + 0.054A^{2/3}$ (Riley Eq.)

FIGURE 5
ALGAL GROWTH RATE

Light Functions (LFNOPT)

* Option 1: Half saturation

$$FL = (1/\lambda D) \ln \left[\frac{KL + I}{KL + Ie^{-\lambda D}} \right]$$

KL = light intensity at which growth rate is 50% of the maximum growth rate.

* Option 2: Smith's Function

$$FL = (1/\lambda D) \ln \left[\frac{I/KL + (1 + (I/KL)^2)^{1/2}}{I/KLe^{-\lambda D} + (1 + (I/KLe^{-\lambda D})^2)^{1/2}} \right]$$

KL = light intensity at which growth rate is 71% of the maximum growth rate.

* Option 3: Steele's Equation

$$FL = \frac{2.718}{\lambda D} \left[e^{-(e^{-\lambda D(I/KL)})} - e^{-I/KL} \right]$$

KL = light intensity at which growth rate is equal to the maximum growth rate.

* Notation

FL = light attenuation factor
λ = extinction coefficient
D = depth
I = surface light intensity

FIGURE 6
ALGAL GROWTH RATE

Growth Rate Attenuation Options (LGROPT)

*Option 1: Multiplicative

$$G = G_{\max} * FL * FN * FP$$

*Option 2: Alternative Nutrient

$$G = G_{\max} * FL * \text{MIN}(FN, FP)$$

*Option 3: Inverse Additive

$$G = G_{\max} * FL * \left[\frac{2}{1/FN + 1/FP} \right]$$

FIGURE 7
DEFAULT THETA VALUES FOR QUAL-2E
EPA/NCASI VERSION

<u>Index</u>	<u>Rate Coefficient</u>	<u>Default Values</u>		<u>Code</u>
		<u>SEMCOG</u>	<u>QUAL-2E</u>	
1	BOD Decay	1.047	1.047	BOD DECA
2	BOD Settling	-	1.024	BOD SETT
3	Reaeration	1.0159	1.024	OXY TRAN
4	SOD Uptake	-	1.060	SOD RATE
5	Organic N Decay	-	1.047	ORGN DEC
6	Organic N Settling	-	1.024	ORGN SET
7	Ammonia Decay	1.047	1.083	NH3 DECA
8	Ammonia Source	-	1.074	NH3 SRCE
9	Nitrite Decay	1.047	1.047	NO2 DECA
10	Organic P Decay	-	1.047	PORG DEC
11	Organic P Settling	-	1.024	PORG SET
12	Dissolved P Source	-	1.074	DISP SRC
13	Algae Growth	1.047	1.047	ALG GROW
14	Algae Respiration	1.047	1.047	ALG RESP
15	Algae Settling	-	1.024	ALG SETT
16	Coliform Decay	1.047	1.047	COLI DEC
17	Non-cons Decay	1.047	1.000	ANC DECA
18	Non-cons Settling	-	1.024	ANC SETT
19	Non-cons Source	-	1.000	ANC SRCE

FIGURE 8
DISSOLVED OXYGEN

Sediment Oxygen Demand Units (SOD)

* Mass per bottom area per time

gram-O₂/m²-day or gm-O₂/ft²-day

* Bottom Area

Bottom Width * Length

* Normal Channels

Bottom Width = Equivalent Width + 2 * Depth

Equivalent Width = Area/Depth
= Flow/(Velocity * Depth)

*Trapezoidal Channels

Bottom Width = Wetted Perimeter

FIGURE 9
INPUT/OUTPUT MODIFICATIONS

New Coding Forms

- * Forthcoming

Local Climatology Data

- * Echo-printed in Steady State (Dynamic Optional)

Descriptive Output for Steady State Convergence

- * Convergence Summary
Algae Growth Rate, Nitrification Inhibition,
Dam Reaeration
- * Temperature Summary - Solar Values
- * Algae Summary - Averaging, Growth and Light
Function Options, TFACT, AFACT, Solar Values

Five Part Final Summary

- * Hydraulics/Geometry
- * Water Quality Coefficients
- * Water Quality Variables
- * Algae Data
- * Dissolved Oxygen Data

Optional Plot of DO and BOD

- * Line Printer Plot

PREDICTING THE REAERATION COEFFICIENT FOR OHIO STREAMS

by Daniel S. Skalsky, Lester D. Fischer, P.E., and Seif Amragy, P.E.
The Ohio Environmental Protection Agency

Introduction

The determination of the reaeration coefficient (K_2) is of critical importance in determining the waste assimilative capacity of streams. This information is essential for effective and efficient planning and implementation of pollution control programs. Comprehensive basin planning, waste load allocation, and the establishment of priorities for construction grants are among the activities which depend upon accurate assessment of a stream's ability to assimilate wastes.

This report was prepared to support the water quality modeling activities of the Ohio Environmental Protection Agency (OEPA). The objectives of this report are to:

1. assess the value of eighteen predictive equations for estimating the reaeration coefficient;
2. examine the relationship between the reaeration coefficient and streamflow; and,
3. make recommendations regarding the determination and use of the reaeration coefficient for waste load allocations.

To achieve the above objectives, an analysis was performed using reaeration data collected during the period from January, 1980 through May, 1983. Data were collected by the U.S. Geological Survey during 1980 - 1982 in cooperation with OEPA. Field studies were continued by OEPA beginning in 1983. Twenty-eight different streams and rivers were studied across the state of Ohio, including a total of 61 distinct reaches. The majority of the study reaches were downstream of wastewater treatment plant discharges.

Methodology

The following sections describe the measurement and calculation techniques used in the preparation and analysis of the data. Additional details of the field techniques and data reduction methods currently employed in reaeration studies are documented in a Reaeration Manual which is being prepared by OEPA. In addition to the reaeration coefficient, streamflow, stream velocity, depth, slope, and cross-section width were also measured for most of the study reaches.

Measurement of K_2 . Reaeration coefficients were measured using the modified tracer technique with ethylene and/or propane as the tracer gas (Rathbun *et al.*, 1975) for all but four determinations of K_2 . These four values were determined using the radioactive tracer technique developed by Tsivoglou *et al.* (1968). All measured K_2 values were converted to values at 20°C using a temperature correction coefficient of 1.024 (Elmore and West, 1961):

$$K_{2,20^{\circ}\text{C}} = K_{2,T}(1.024)^{(20-T)} \quad (1)$$

where T is the measured stream temperature in degrees C.

Most of the rivers in the data set have field K_2 values available for one or two reaches, although a few, such as the Great Miami and Scioto rivers, have measurements for additional reaches. When both ethylene and propane K_2 values were available, the average of the two values was used in the analysis. A comparison of ethylene and propane results is presented in Figure 1. It is clear that the results obtained using the two hydrocarbon gases are not always in complete agreement with one another.

The data were edited and in a few studies were judged unreliable because of unusual field conditions, data inconsistencies, or questions regarding the validity of the assumptions involved in the measurement procedure. All negative K_2 values which were obtained were associated with unreliable data and were thus deleted from the analysis. Inadequate mixing of dye and gas tracers in the cross-section was noted as a possible cause of unreliable field data. Poor mixing was suspected when an irregular dye concentration versus time curve was observed at the first sampling site or when an extreme loss or apparent gain of dye mass was observed between sites.

Fifty-four valid determinations of K_2 were included in the final data set, upon completion of the above editing procedures. Reaeration coefficient values ranged from 1.05 to 48.0 per day at 20 degrees C (base e). Two studies had K_2 values greater than 14.0, both of which involved rapid changes in elevation (i.e., a cascade or low head dam).

Velocity Measurement. Velocity was calculated by dividing the length of a given reach by the time of travel of the dye peak through the reach. Reach length was determined from topographic maps. A comparison of velocities calculated using peak and mean times of passage for 10 studies showed that the two methods differed by approximately 11 percent.

The velocities measured in the study reaches ranged from .054 to 2.33 feet per second. Eighty-five percent of the reaches studied had velocities less than one foot per second.

Depth Calculation. Average velocity and flow measurements were used in conjunction with one to four stream top width measurements obtained within a given reach to calculate the average depth using the equation:

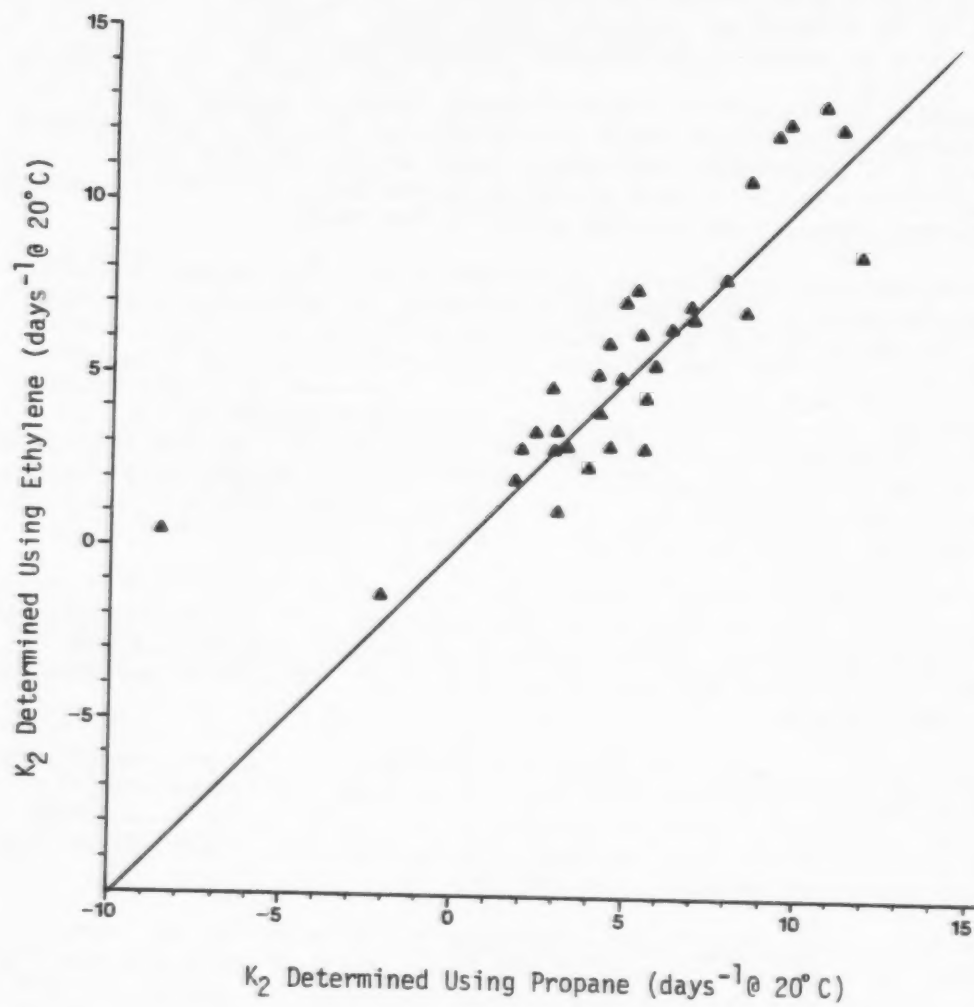


Figure 1: Comparison of K_2 Values Obtained Using Ethylene and Propane Tracer Gases.

$$H = \sum_{i=1}^n \left[\frac{Q}{V W_i} \right] \frac{L_i}{L} \quad (2)$$

where

- H = average depth of flow (ft);
- Q = discharge (cfs);
- V = time of travel velocity (peak to peak, ft/s);
- L = length of reach (ft);
- W_i = width of subreach i (ft);
- L_i = length of subreach i (ft); and,
- n = number of homogeneous subreaches.

In Equation 2, the term in brackets may be viewed as a local depth, indicating the calculated depth at a particular location along the reach where a width measurement was taken. Local depths are then weight averaged within a given reach according to the length of their respective subreaches, yielding the average depth for the reach.

A comparison of average depths calculated in the above manner to those calculated using a single average width value in conjunction with velocity and flow was performed. The results yielded by the two methods were essentially identical for the data examined, with less than 2 percent difference between methods. The reason for the close agreement between the two calculation procedures may be due in part to the limited resolution of the width, velocity, and flow data. In the case where only 1 width measurement was available, the two calculation procedures yielded identical results. Calculated average depths ranged from 0.2 to 7.3 feet, and only two reaches had average depths which exceeded 4.0 feet.

Slope Measurement. Water surface slope was surveyed directly in over 90 percent of the reaches studied and was estimated from topographic maps in the remaining reaches. Slopes in the study reaches ranged from 0.63 to 72.18 feet per mile, less than 25 percent of the measured slopes exceeded 10.0 feet per mile.

Streamflow Measurement. Standard U.S. Geological Survey procedures were used to measure streamflow with Price AA or Pygmy current meters (refer to Buchanon and Somers, 1969 for details of this technique). Flow ranged from 0.1 to 1470 cfs among reaches. Approximately 80 percent of the study reaches had flows between 1 and 300 cfs.

Analysis of Predictive Equation Performance. A comparison was made between observed reaeration rates and those predicted using the equations shown in Table 1. The data base used in the derivation and/or validation of each equation is also noted in Table 1. Table 2 summarizes a select group of these data sets in terms of the range of several common hydraulic variables.

No attempt was made to segregate the available data according to the range of the original data used in the derivation and/or validation of the equations examined. Segregation of the data in this manner makes a direct comparison of performance difficult, since the data set used for the

Table 1. Equations for Predicting K_2 .

1. Bansal (1973). Data from: Churchill et al., Langbein - Durum, and O'Connor - Boddins.

$$K_2 = 4.666 V^{0.6} H^{-1.4}$$
2. Bennett - Rathbun I (1972). Data from: Churchill et al., Krenkel-Orlob, Negeleacu-Rojanski, O'Connor-Dobbins, Owens et al., Thackston-Krenkel, Tsivoglou-Neal (pre-1968 data only).

$$K_2 = 106.1 V^{0.413} S^{0.273} H^{-1.408}$$
3. Bennett - Rathbun II (1972).

$$K_2 = 20.17 V^{0.607} H^{-1.689}$$
4. Cadwallader - McDonnell (1969). Data from: Churchill et al., Owens et al.,

$$K_2 = 336.8 (VS)^{0.5} H^{-1.0}$$
5. Churchill et al. I (1962). Authors' data.

$$K_2 = 0.0944 V^{2.361} H^{-2.753} S^{-0.669}$$
6. Churchill et al. II (1962). Authors' data.

$$K_2 = 11.57 V^{0.969} H^{-1.673}$$
7. Covar (1976) - combined the equations of O'Connor and Dobbins (1958), Churchill et al. II (1962), and Owens et al. (1964).

$$K_2 = \begin{matrix} 12.81 & V^{0.5} & H^{-1.5} & V < 2-3 \text{ ft/s;} & H > 2 \text{ ft} \\ 11.6 & V^{0.969} & H^{-1.673} & V > 2-3 \text{ ft/s;} & H > 2 \text{ ft} \\ 21.7 & V^{0.67} & H^{-1.85} & & H < 2 \text{ ft} \end{matrix}$$
8. Dobbins (1965). Data from: Churchill et al., Krenkel-Orlob, and O'Connor-Dobbins.

$$K_2 = 127.7 \left[\frac{(1 + F^2) (VS)^{0.375}}{(0.9 + F)^{1.5} H} \right] \coth \left[\frac{5.11 (VS)^{0.125}}{(0.9 + F)^{0.5}} \right]$$
9. Foree (1979). Author's data.

$$K_2 = (0.56 + 6786 S^{1.15}) q^{0.25}$$

if $q > 1.0$ use $q = 1.0$
if $q < 0.05$ use $q = 0.05$
10. Isaacs - Gaudy (1968). Authors' data.

$$K_2 = 8.61 V H^{-1.5}$$

Table 1. Equations for Predicting K_2 . (Continued)

11. Krenkel - Orlob (1963). Authors' data.
 $K_2 = 234. (VS)^{0.408} H^{-0.66}$
12. Langbein - Durum (1967). Data from: Churchill et al.,
 Krenkel-Orlob, and O'Connor-Dobbins.
 $K_2 = 7.60 V H^{-1.33}$
13. Negulescu - Rojanski (1969). Authors' data.
 $K_2 = 10.92 (V/H)^{0.85}$
14. O'Connor - Dobbins (1958). Authors' data.
 $K_2 = 12.81 V^{0.5} H^{-1.5}$
15. Padden - Gloyna (1971). Authors' data.
 $K_2 = 6.87 V^{0.703} H^{-1.054}$
16. Parkhurst - Pomeroy (1972). Authors' data.
 $K_2 = 48.39 (1 + 0.17 F^2) (VS)^{0.375} H^{-1.0}$
17. Thackston - Krenkel (1969). Authors' data.
 $K_2 = 24.94 (1 + F^{0.5}) U^* H^{-1.0}$
18. Tsiavoglou - Neal (1976). Authors' data.
 $K_2 = C V S$
 if $1 < Q < 10$ cfs, $C = 9500$
 if $10 \leq Q \leq 25$ cfs, $C = 6860$
 if $25 < Q$, $C = 4650$

where

K_2 = reaeration rate coefficient at 20°C (base e,
 day^{-1});
 V = average velocity (ft/s);
 H = average hydraulic depth (ft);
 S = slope of the energy gradient (ft/ft);
 F = Froude number = $V (32.2 H)^{-0.5}$;
 q = specific discharge (cfs) / mi^2 ; and,
 U^* = average shear velocity (ft/s) = $(32.2 H S)^{0.5}$.

Table 2. Summary of Selected Reaeration Data Bases (adopted from Bennett and Rathbun, 1972).

REFERENCE	NUMBER OF DATA POINTS	FLOW (cfs)		RIVER	FLUME	K_2 (days ⁻¹)		DEPTH (ft)		VELOCITY (ft)		SLOPE (ft/ft)	
		MIN	MAX			MIN	MAX	MIN	MAX	MIN	MAX	MIN	MAX
O'Connor-Dobbins, 1958	38	-	-	X		0.04	13.36	0.90	37.00	0.19	4.20	.000027	.003600
Churchill et al., 1962	30	952	17,300	X		0.53	12.80	2.10	11.40	1.80	5.00	.000126	.002351
Krenkel-Orlob, 1963	58	-	-		X	24.5	266.0	0.08	0.20	0.13	2.14	.000750	.024000
Owens et al., 1964	32	2.70	2.16	X		0.71	11.32	0.39	2.44	0.13	1.83	.000156	.010600
Isaacs-Gaudy, 1968	52	-	-		X	0.51	12.80	2.12	11.41	1.85	5.00	-	-
Thackston-Krenkel, 1969	40	-	-		X	17.64	149.0	0.04	0.23	0.37	2.32	.000650	.020400
Negelescu-Rojanski, 1969	8	-	-		X	19.90	43.07	0.16	0.49	0.66	1.90	-	-
Parkhurst-Pomeroy, 1972	74	0.4	8.8		Sewers	-	-	0.26	1.59	1.88	7.88	.000160	.009800
Tsivoglou-Neal, 1976	605	0.2	3,000	X		0.08	360.0	-	-	-	2.20	.000095	.056818
Free, 1979	42	0.3	410	X		0.34	37.75	-	-	0.06	0.94	0.0	.010890

evaluation would be different for each equation. The current analysis is a simple assessment of the performance of several published equations for predicting K_2 using the available data on Ohio streams.

The first step in comparing predicted and observed K_2 values was the examination of scatterplots of observed versus predicted values. The relationship between predicted and observed values included a great deal of scatter, as shown in Figure 2.

The data set was then subdivided into groups based on the slope of the energy gradient and streamflow. The four classes are defined in Table 3. Slope and flow were used to classify streams because they reflect hydraulic regimes concisely. Both variables are normally measured or estimated as part of a modeling project.

Table 3. Description of Data Classes.

Data Class	Number of Data Points	Slope (ft./mi.)	Flow (cfs)
1	11	< 3	-
2	15	3 - 10	≤ 30
3	13	3 - 10	> 30
4	14	> 10	-

The above classification scheme resulted in 11-15 measured K_2 values in each class. An examination of the percentage errors of each equation was conducted to determine the best equations in each class, based on four criteria:

1. the percentage of relative errors with absolute values less than 50 percent;
2. the percentage of relative errors less than -80 percent;
3. the percentage of relative errors greater than 100 percent; and,
4. the percentage of overpredictions and underpredictions.

Although the numerical values used in the above criteria are somewhat arbitrary, the performance criteria provide a great deal of information about the error distribution associated with each predictive equation. The first criterion reflects the percentage of reasonably accurate predictions and was weighted most heavily in the analysis. The second and third criteria indicate extreme negative and positive errors (i.e., underestimates and overestimates, respectively). The final criterion indicates whether or not the error distribution is centered about zero.

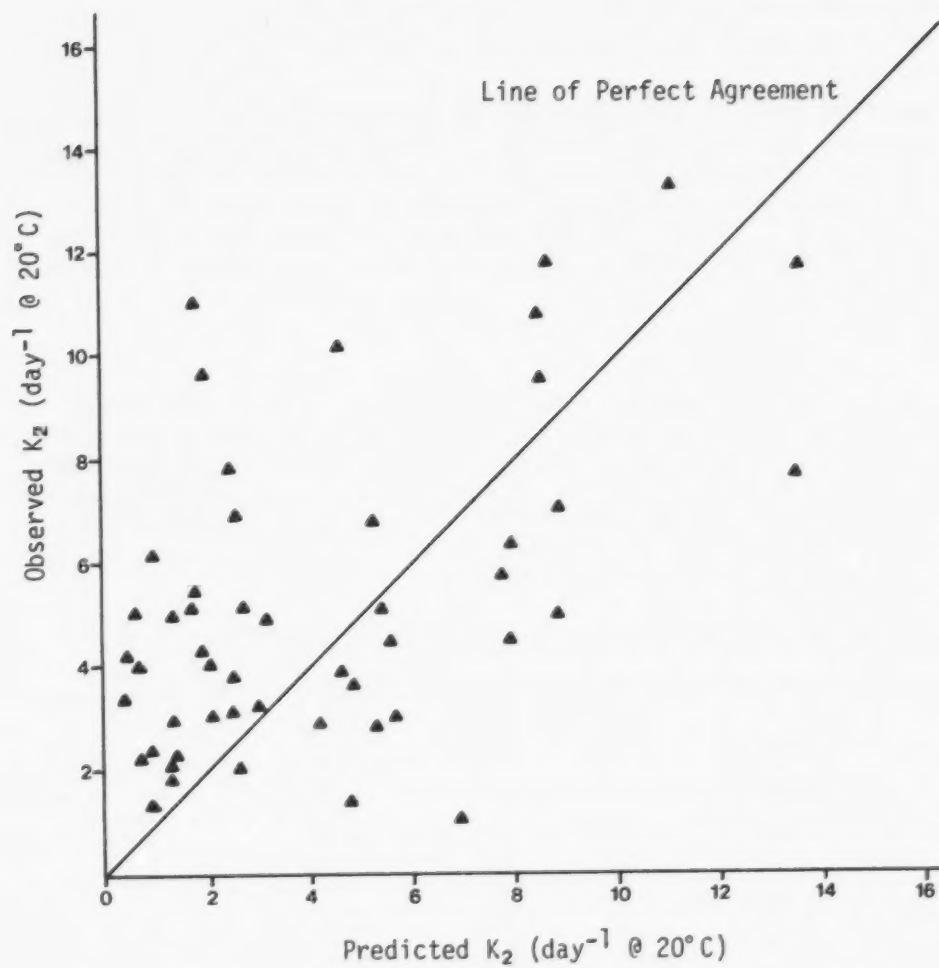


Figure 2: Observed K_2 Versus Tsivoglou-Neal (1976) Predictions.

Percent relative error was calculated using the following equation:

$$E = \frac{(K_2' - K_2)}{K_2} 100 \quad (3)$$

where

E = percent relative error;

K_2 = measured K_2 at 20°C (base e, day⁻¹); and,

K_2' = predicted K_2 at 20°C (base e, day⁻¹).

Percent relative error (E) was selected as the basic measure of agreement between observed and predicted reaeration coefficients for two reasons. First, this approach maintains case by case detail which is useful in guiding the examination of outliers. Second, percent relative error provides a measure of predictive ability in terms which are similar to those used to define the range of K_2 values used in wasteload allocation sensitivity studies.

Results

Variation of K_2 With Streamflow. The variation of K_2 with streamflow is important, since reaeration studies are rarely conducted under exactly the same hydrologic conditions which are assumed in water quality models for waste load allocation purposes. However, there is a general lack of information available on the variation of reaeration with streamflow in Ohio. The data which are available are presented in Table 4 and in each case, K_2 increased with increasing flow.

Table 4. Variation of Reaeration Rate with Varying Flow Conditions.

Stream	Reach	Velocity (ft/s)	Depth (ft)	Flow (cfs)	K_2 (day ⁻¹ , 20°C)
Pawpaw Cr.	1	.210	1.0	4.55	3.73
	1	.387	1.1	9.62	4.94
N. F. Licking R.	1	.10	0.6	8.67	2.04
	1	.22	0.9	19.20	3.09
	2	.15	0.6	8.86	2.23
	2	.28	0.9	19.80	3.32

To estimate the behavior of K_2 with flow variations in the absence of field measurements velocity versus flow and depth versus flow relationships may be used in conjunction with a predictive equation for K_2 (Zogorski and Faust, 1973).

Comparison of Predictive Equations. The performance of each equation with respect to the four performance criteria in each data class (1-4) is displayed graphically in Appendix A, Figures A-1 through A-4. The predictive equations which yielded the best results in each data class are shown in Table 5.

Table 5. Preferred Predictive Equations.

Data Class	Preferred Predictive Equation(s)
1	Krenkel-Orlob; Negelescu-Rojanski
2	Parkhurst-Pomeroy
3	Thackston-Krenkel
4	Tsivoglou-Neal

The reaeration coefficients predicted by the Krenkel-Orlob (1963) and Negelescu-Rojanski equations were closest to the observed values in Class 1 (slopes less than 3 ft/mi). Over 70 percent of the data in this class yielded relative errors (E) less than 50 percent in absolute value for these equations. Less than 10 percent of the data fell into either of the extreme value categories (i.e., $E \leq -80$; $E \geq 100$). A complete summary of the relative errors for all predictive equations when water surface slope was less than 3 ft./mi. is shown in Appendix A, Figure A-1. Scatterplots of observed versus predicted K_2 values for these two equations are shown in Appendix B, Figures B-1 and B-2.

It is interesting that the Krenkel-Orlob equation performed well in Data Class 1. Previous reviewers have noted the unusually high values of K_2 which were obtained in the laboratory experiments upon which this equation is based (Thackston and Krenkel, 1969; Wilson and Macleod, 1974). One possible explanation for this result is that low-slope streams in Ohio are generally sluggish and may include large pooled reaches. In such streams it is conceivable that factors which are not represented adequately by velocity and depth are more significant in determining the rate of reaeration (e.g., wind).

A second possible reason for the seemingly high K_2 values in low-slope streams is that the modified tracer technique may overestimate reaeration for sluggish reaches. The data used in this analysis indicate that field problems and measurement errors are more common on such streams. However, a tendency to overestimate the reaeration coefficient cannot be determined from the available data. Other researchers have demonstrated that the modified tracer technique may yield larger K_2 values than the radioactive tracer method in some cases (Grant and Skavronneck, 1980; Whittemore, 1982).

The best agreement with the experimental data in Class 2 (slope of 3-10 ft./mi. and flow less than or equal to 30 cfs) was observed for the Parkhurst - Pomeroy equation (1972), followed by the equations of Bansal (1973), Langbein - Durum (1967), and Padden - Gloyna (1971). These equations produced absolute relative errors less than 50 percent for approximately 70 percent of the Class 2 data. None of these equations produced any extreme errors, as shown in Appendix A, Figure A-2. Scatterplots for observed versus predicted K_2 values for these equations are shown in Appendix B, Figures B-3 through B-6.

Only the Thackston - Krenkel equation (1969) performed well in Class 3 (slope of 3-10 ft./mi. and flow larger than 30 cfs), as shown in Appendix A, Figure A-3. Sixty-nine percent of the relative errors yielded by this equation in Class 3 data were less than 50 percent in absolute value while 15 percent of relative errors fell in the extreme categories. A scatterplot and associated linear regression statistics for observed versus predicted K_2 values for the Thackston - Krenkel equation are shown in Appendix B, Figure B-7.

In Class 4 (slopes greater than 10 ft./mi.) the equations of Foree (1979), Negelescu-Rojanski, O'Connor-Dobbins, Parkhurst-Pomeroy (1972), and Tsivoglou-Neal (1976) performed well in terms of relative errors as shown in Appendix A, Figure A-4. Each yielded errors less than 50 percent in absolute value for more than 60 percent of the data. The Tsivoglou-Neal (1976) equation is clearly preferable because it had the most predictions within 50 percent relative error, yielded no extreme errors, and the error distribution was centered about zero (i.e., the equation showed no tendency to over or underpredict K_2 in this data class). Scatterplots for the Tsivoglou-Neal (1976) and Parkhurst-Pomeroy (1972) equations are shown in Appendix B, Figures B-8 and B-9. It appears that the Parkhurst-Pomeroy equation could be improved by refining the leading coefficient in the equation to better fit the observed data.

In general, the analysis showed that the performance of predictive equations varied considerably depending on the type of streams to which they are applied. Figure 3 shows a scatterplot of observed versus K_2 values predicted using each of the preferred predictive equations in their respective class (the Krenkel-Orlob equation was arbitrarily selected for use in Data Class 1). It is surprising that four of the five preferred equations were not developed using extensive stream data, but were based primarily on flume data (Negelescu and Rojanski, 1969; Krenkel and Orlob, 1963; and, Thackston and Krenkel, 1969) and sewer data, in the case of Parkhurst and Pomeroy (1972). This finding supports the use of these equations, which have not gained wide acceptance because of concern regarding possible differences between the hydraulic characteristics of flumes and natural streams.

Conclusions

The following conclusions are made based on the preceeding analysis of reaeration in Ohio streams:

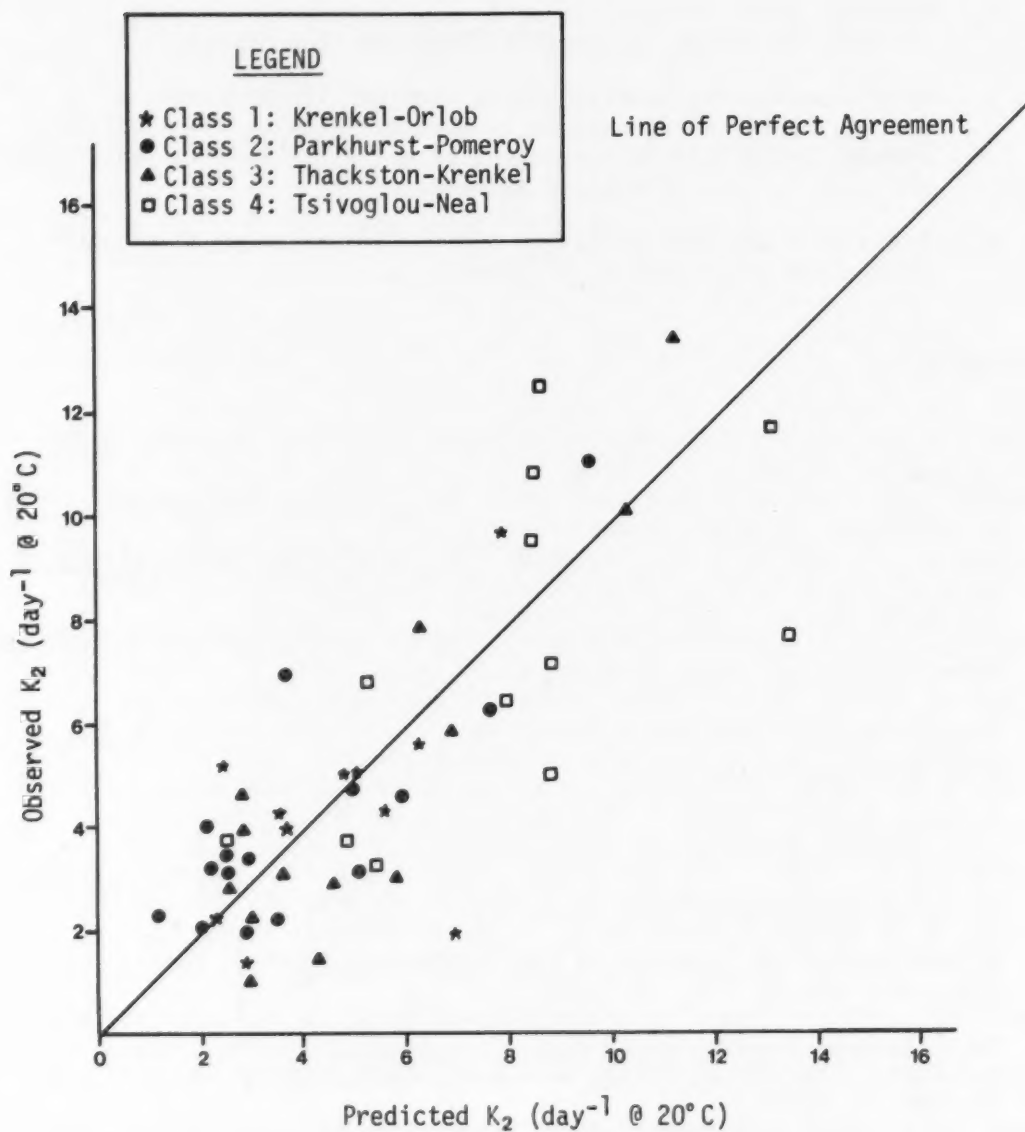


Figure 3: Comparison of Observed Versus Predicted K_2 Values Using a Preferred Equation in Each Data Class.

1. Field determinations of K_2 are preferable to predictive equations when the stream analysis is sensitive to K_2 .
2. Relative errors in predicting K_2 may be reduced by using selected equations in specific slope and flow classes.
3. Model sensitivity studies should consider larger ranges of K_2 than currently used, especially when K_2 is below 2. Absolute ranges should also be considered (e.g., 0.1 - 3.0) in sensitivity analyses for low reaeration streams.
4. Additional data are needed to define the variation of K_2 with streamflow in a given river reach.

References

- Bansal, M. K., 1973. "Atmospheric Reaeration in Natural Streams" Water Research V. 7, No 5, pp. 769-82.
- Bennett, J. P., and R. E. Rathbun, 1972. Reaeration in Open Channel Flow, US Geological Survey Professional Paper 737.
- Buchanon, T. J. and W. P. Somers, 1969. Discharge Measurements at Gaging Stations, Techniques of Water Resources Investigations of the US Geological Survey, Book 3, Chapter A8.
- Cadwallader, T. E., and A. J. McDonnell, 1969. "A Multivariate Analysis of Reaeration Data," Water Research V. 3, pp. 731-742.
- Churchill, M. A., H. L. Elmore, and R. A. Buckingham, 1962. "The Prediction of Stream Reaeration Rates," Journal of the Sanitary Engineering Div., ASCE V. 88, No. SA4, pp. 1-46.
- Covar, A. P., 1976. "Selecting the Proper Reaeration Coefficient for Use in Water Quality Models," Proceedings of USEPA Conference on Environmental Modeling and Simulation, pp. 340-343.
- Dobbins, W. E., 1965. "BOD and Oxygen Relationships in Streams," Journal of the Sanitary Engineering Div., ASCE V. 91, No. SA5, pp. 49-55.
- Elmore, H. L. and W. F. West, 1961. "Effects of Water Temperature on Stream Reaeration," Journal of the Sanitary Engineering Division, ASCE, Vol. 85, No. SA4, pp. 59-71.
- Foree, E.G., 1979. "Low-Flow Reaeration and Velocity Characteristics of Small Streams," Symposium on Reaeration Research, Proceedings Hydraulics Division Specialty Conference ASCE, October 28, 1975. pp. 185-209.
- Grant, R. S. and S. Skavroncek, 1980. Comparison of Tracer Methods and Predictive Equations for Determination of Stream-Reaeration Coefficients on Three Small Streams in Wisconsin. US Geological Survey Water Resources Investigation 80-19.
- Isaacs, W. P. and A. F. Gaudy, 1968. "Atmospheric Oxygenation in a Simulated Stream," Journal of the Sanitary Engineering Division, ASCE, Vol. 94, No. SA2, pp. 319-344.
- Krenkel, P. A. and G. T. Orlob, 1963. "Turbulent Diffusion and the Reaeration Coefficient," American Society of Civil Engineers Transactions V. 128, pp. 293-334.

- Langbein, W. B. and Will Durum, 1967. The Aeration Capacity of Streams, US Geological Survey Circular 542.
- Negelescu, M. and V. Rojanski, 1969. "Recent Research to Determine Reaeration Coefficient," Water Research V. 3, No. 3, pp. 189-202.
- O'Connor, D.J. and W. E. Dobbins: 1958, "Mechanism of Reaeration in Natural Streams," American Society of Civil Engineers Transactions V. 123, pp. 641-684.
- Owens, M., R. W. Edwards, and J. W. Gibbs, 1964. "Some Reaeration Studies of Streams," Int. Journal of Air and Water Pollution V. 8, pp. 469-486.
- Padden, T. J. and E. F. Gloyna, 1971. Simulation of Stream Processes in a Model River, University of Texas, Austin, Report No E11E-70-23, CRWR-72.
- Parkhurst, J. D. and R. D. Pomeroy, 1972. "Oxygen Absorption in Streams," Journal of the Sanitary Engineering Division, ASCE V. 98, No. SA1, pp. 101-124.
- Rathbun, R. E., D. J. Shultz, and D.W. Stephens, 1975. Preliminary Experiments with a Modified Tracer Technique for Measuring Stream Reaeration Coefficients, US Geological Survey Open File Report 75-256.
- Thackston, E. L. and P. A. Krenkel, 1969, "Reaeration Prediction in Natural Streams," Journal of the Sanitary Engineering Division, ASCE V. 95, No. SA1, pp. 65-94.
- Tsivoglou, E. C., J. B. Cohen, S.D. Sheaver, and P. J. Godsil, 1968 "Tracer Measurement of Stream Reaeration II: Field Studies," Journal of the Water Pollution Control Federation V. 40, No. 2, pp. 285-305.
- Tsivoglou, E. C. and L. A. Neal, 1976 "Tracer Measurement of Reaeration: III. Predicting the Reaeration Capacity of Inland Streams," Journal of the Water Pollution Control Federation V. 48, No. 12, pp. 2669-88.
- Whittemore, R.C., 1982. A Comparison of Reaeration Estimation Techniques for the Ouachita River Basin, NCASI Technical Bulletin No. 375.
- Wilson, G. T. and N. Macleod, 1974. "A Critical Appraisal of Empirical Equations and Models for the Prediction of the Coefficient of Reaeration of Deoxygenated Water," Water Research V. 8, No. 6, pp. 341-366.
- Zogorski, J. S. and S. D. Faust, 1973. "Atmospheric Reaeration Capacity of Streams, Part I: Critical Review of Methods Available to Measure and Calculate the Atmospheric Reaeration Rate Constant," Environmental Letters V. 4, No. 1, pp. 35-59.

Appendix A: Summary of Predictive Equation Performance

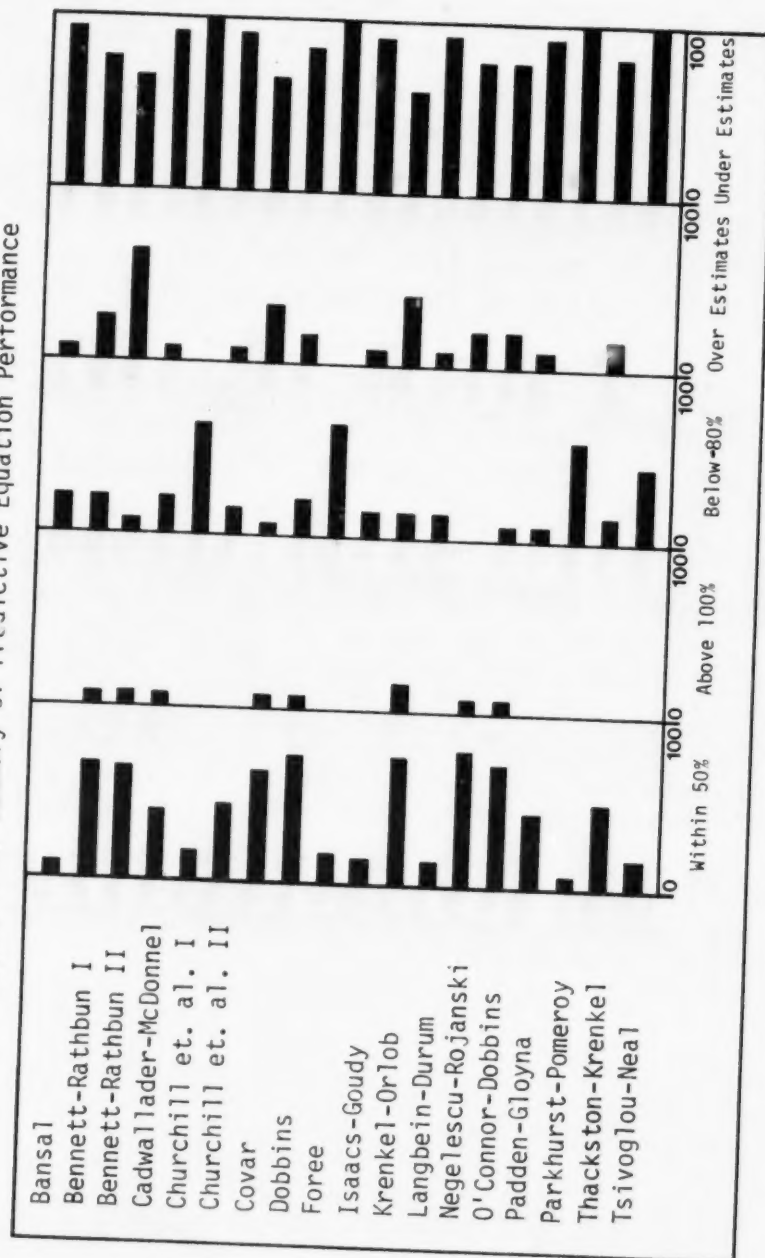


Figure A-1: Summary of Percent Relative Error of Predictive Equations For Slope Less Than 3 (ft./mi.).

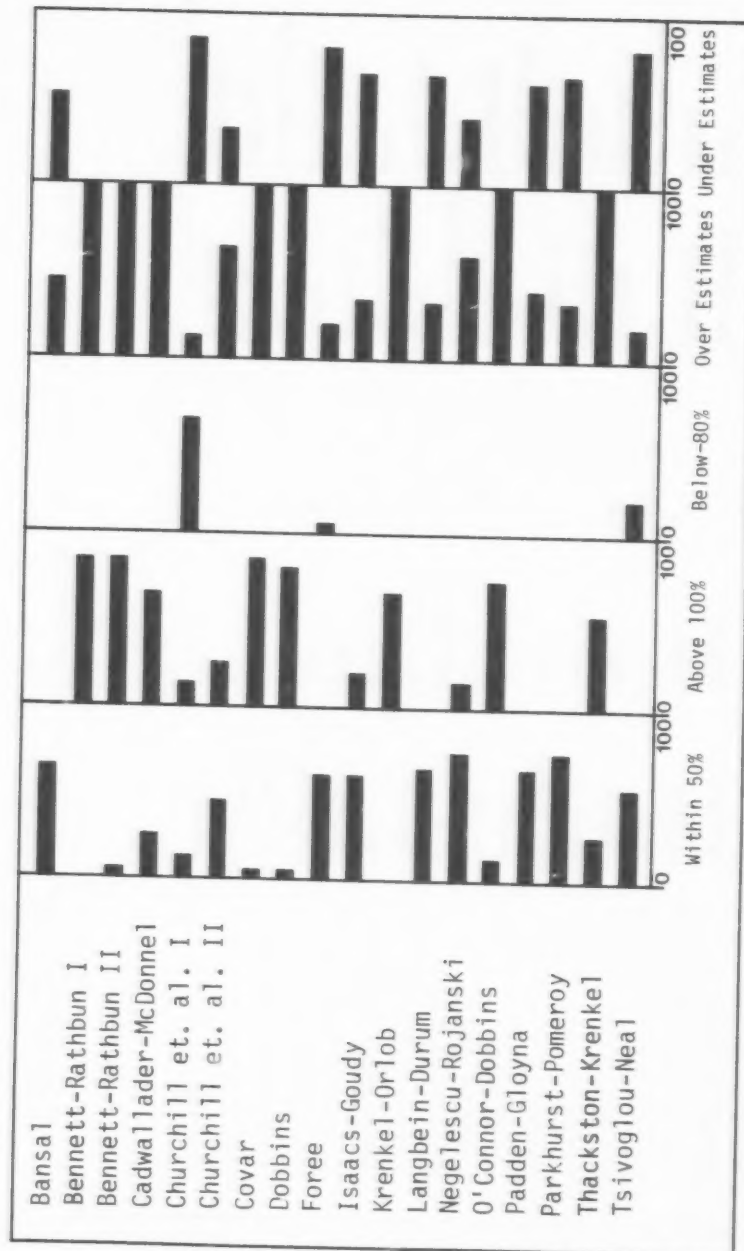


Figure A-2: Summary of Percent Relative Error of Predictive Equations For Slope From 3 Through 10 (ft./mi.); Flow Less Than or Equal to 30 (cfs).

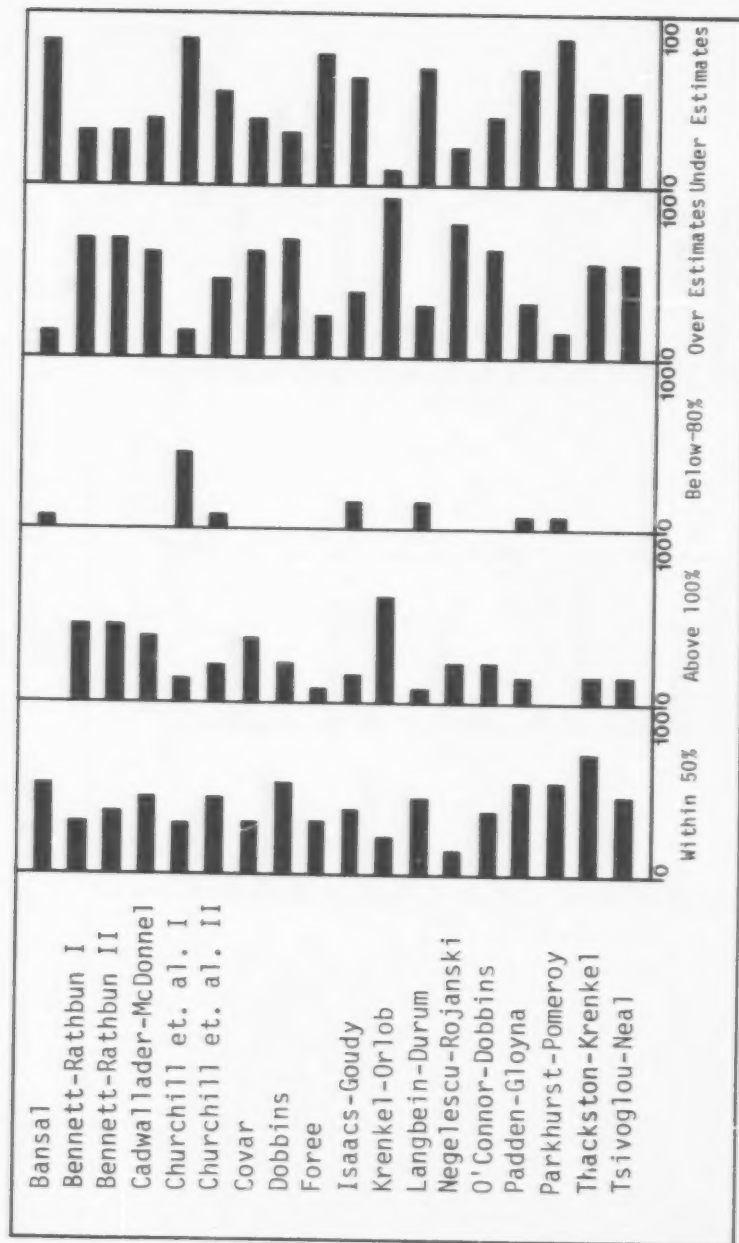


Figure A-3: Summary of Percent Relative Error of Predictive Equations For Slope From 3 Through 10 (ft./mi.); Flow Greater Than 30 (cfs).

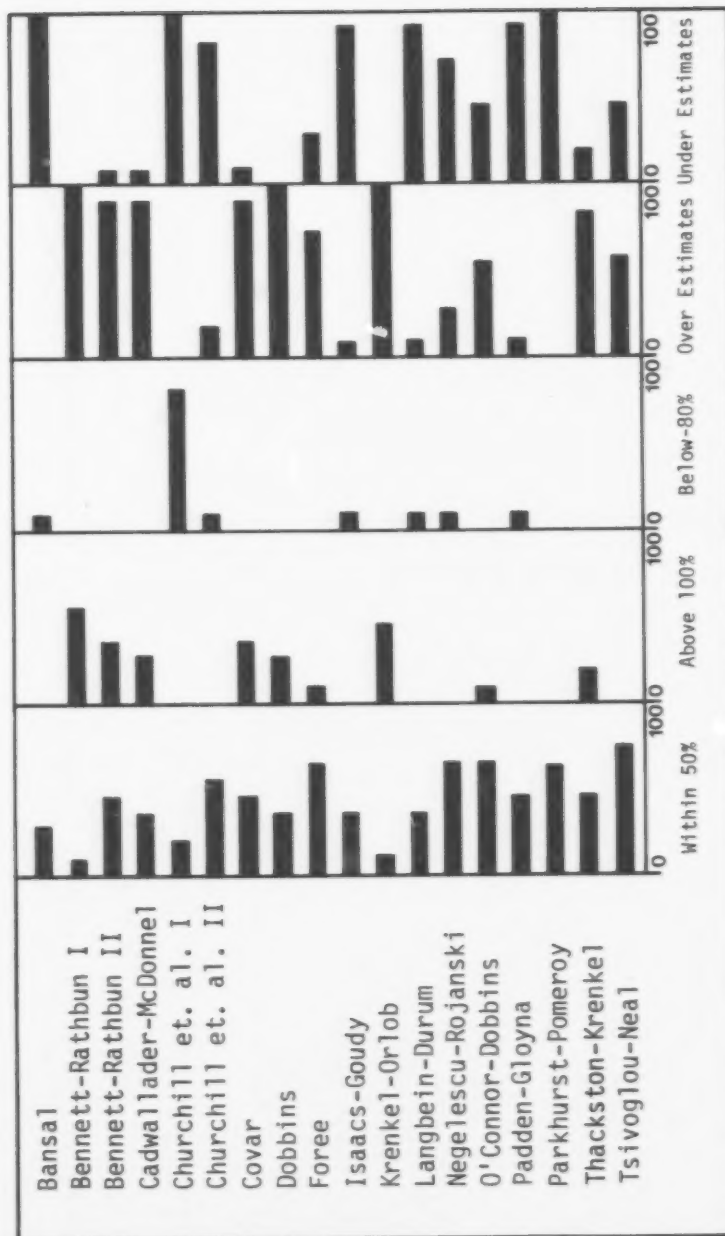


Figure A-4: Summary of Percent Relative Error of Predictive Equations For Slope Greater Than or Equal to 10 (ft./mi.).

Appendix B: Plots of Observed Versus Predicted Reaeration Coefficients

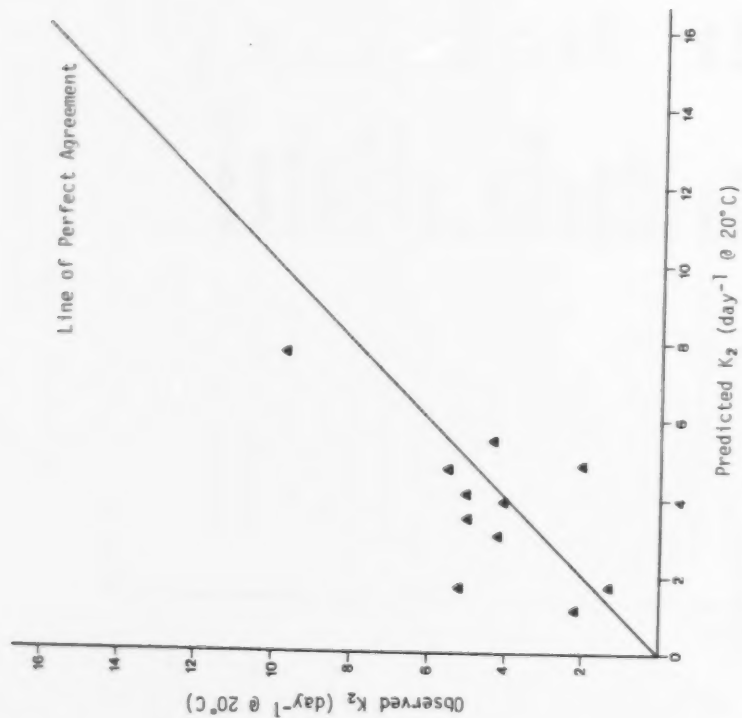


Figure B-1: Observed K_2 Versus Negelescu-Rojanski (1969) Predictions for Slope Less Than 3 (ft./mi.)

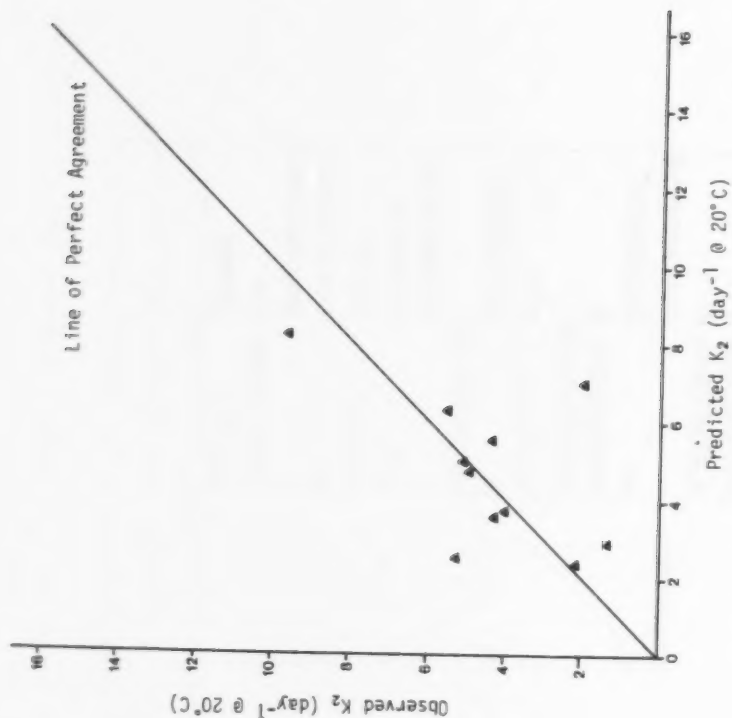


Figure B-2: Observed K_2 Versus Krenkel-Orlob (1963) Predictions for Slope Less Than 3 (ft./mi.)

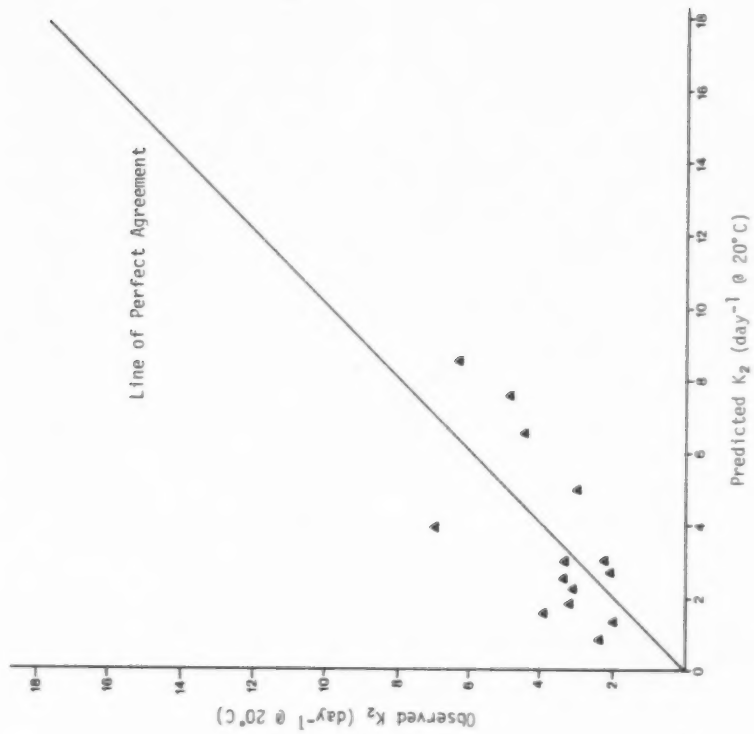


Figure B-4: Observed K_2 Versus Bansal (1973) Predictions for Slope From 3-10 (ft./mi.); Flow Less Than or Equal to 30 (cfs)

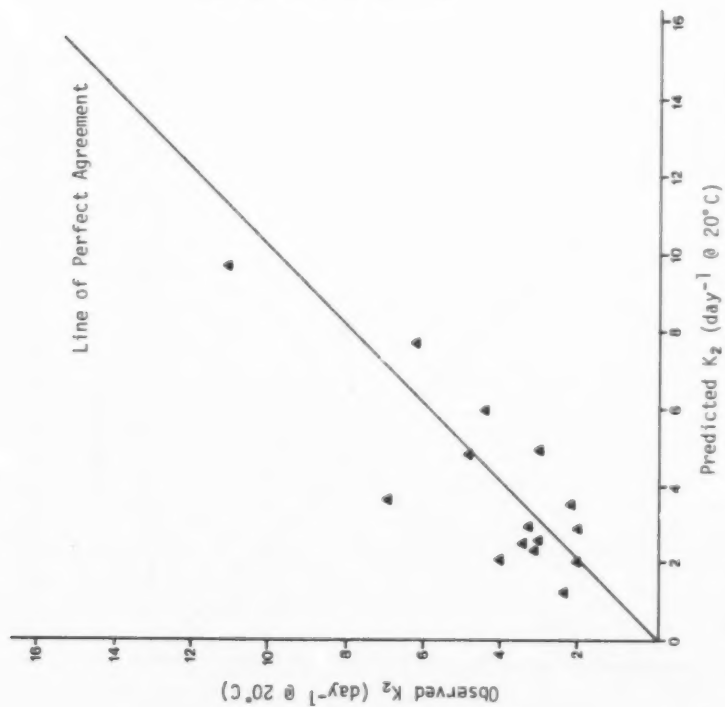


Figure B-3: Observed K_2 Versus Parkhurst-Pomeroy (1972) Predictions for Slope From 3-10 (ft./mi.); Flow Less Than or Equal to 30 (cfs)

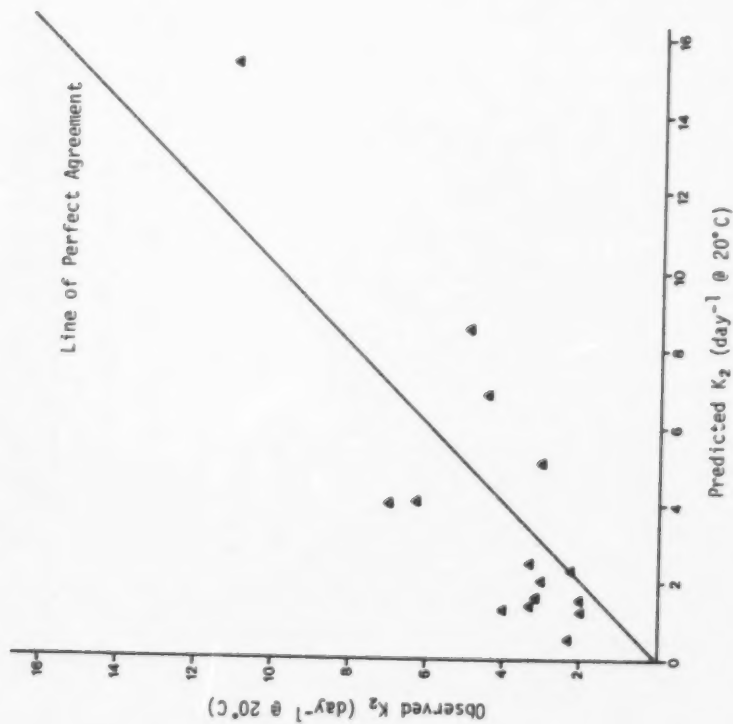


Figure B-5: Observed K_2 Versus Langbein-Durum (1967)
Predictions for Slope 3-10 (ft./mi.); Flow Less
Than or Equal to 30 (cfs)

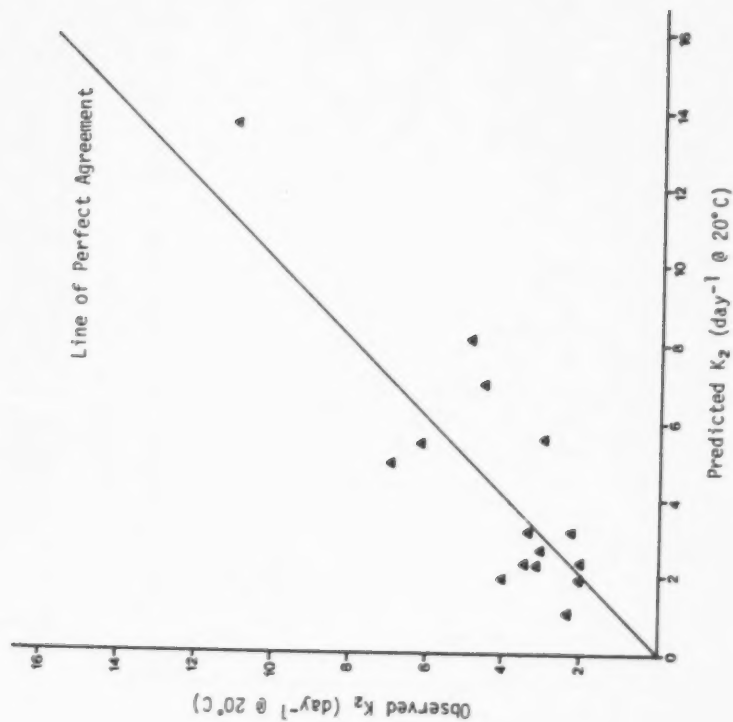


Figure B-6: Observed K_2 Versus Padden-Gloyna (1971)
Predictions for Slope From 3-10 (ft./mi.); Flow
Less Than or Equal to 30 (cfs)

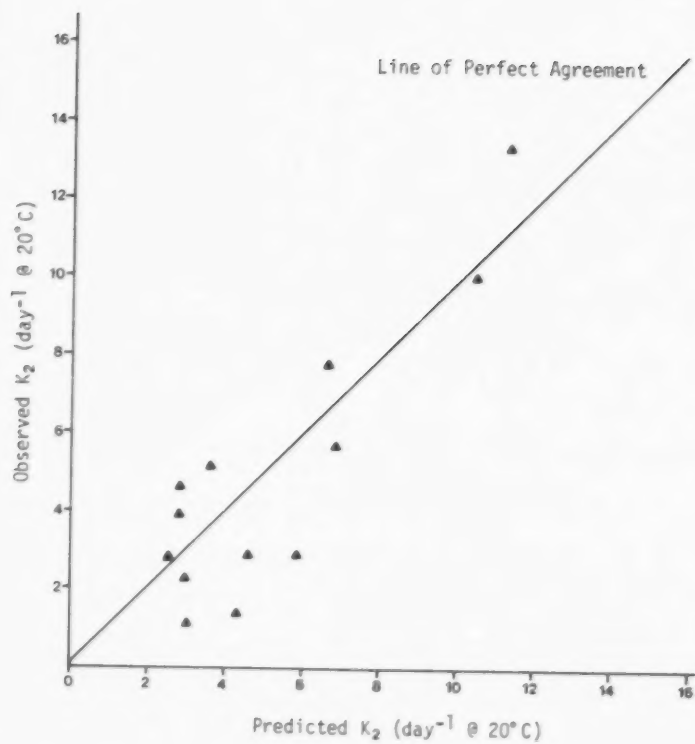


Figure B-7: Observed K_2 Versus Thackston-Krenkel (1969)
Predictions for Slope From 3-10 (ft./mi.); Flow
Greater Than 30 (cfs)

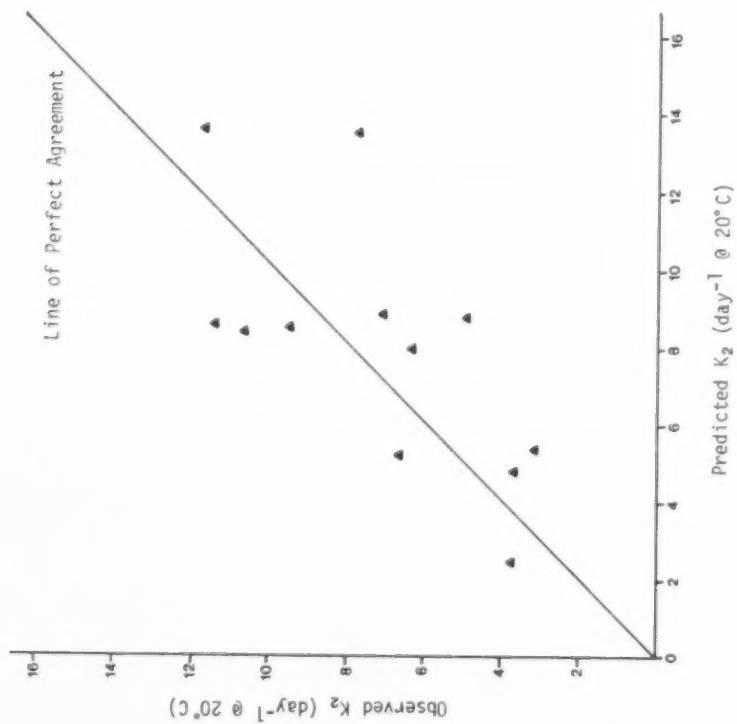


Figure B-8: Observed K_2 Versus Tsivoglou-Neal (1976) Predictions for Slope Greater Than or Equal to 10 (ft./mi.)

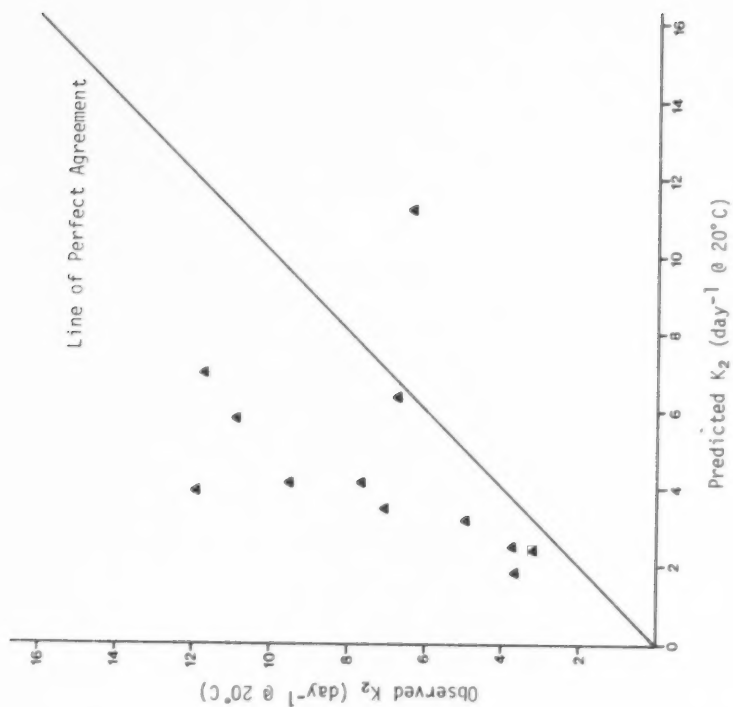


Figure B-9: Observed K_2 Versus Parkhurst-Pomeroy (1972) Predictions for Slope Greater Than or Equal to 10 (ft./mi.)

WATER TEMPERATURE MODELING: A PRACTICAL GUIDE

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ABSTRACT

This paper is a review of techniques for the computation of water temperature in surface water bodies. Emphasis is placed on the practical aspects of applying water temperature models. The discussion focuses particularly upon the problems of obtaining and adapting the meteorological data required as input for surface heat transfer calculations.

INTRODUCTION

Computations of water temperature are employed to determine the environmental impacts of thermal discharges, to evaluate the performance of cooling ponds used to dispose of waste heat from power plants, or to evaluate the hydrothermal characteristics of water bodies in general. They are an essential part of the design of waste heat disposal structures and systems, and in the assessment of environmental effects of waste heat disposal.

The calculation of the water temperature of a surface-water body is based upon a heat balance similar to the water or mass balances employed in water quality modeling. The heat balance determines the change in the heat stored as equal to the influx of heat less the outflux of heat. The heat stored in a quantity of water is manifested by its temperature according to the following proportionality:

$$H = V \rho c_p T \quad (1)$$

where

H	is the total heat content of the volume of water (Btu),
V	is the volume of water (ft ³),
ρ	is the density of water (lb/ft ³),
c_p	is the specific heat of water (Btu/lb/°F), and
T	is the water temperature (°F).

The fluxes of heat to or from a water body may be classified as follows:

1. Inflowing and outflowing water. These represent heat fluxes by virtue of their temperature. The calculation of these fluxes is a straightforward extension of Equation 1.
2. Heat conduction to or from the earth through the bottom of the water body. This is a minor component of the heat balance that is usually neglected.
3. Heat transfer through the water surface. This is a major component of the heat balance and the primary topic of this paper.

The calculation of water temperature by a computer model is based upon bookkeeping these fluxes and determining the consequent change in temperature.

As with water quality modeling, one may employ various assumptions concerning the internal distribution of heat or temperature within a water body. The simplest assumption is that the water body is isothermal (fully-mixed). This is often inaccurate for deep lakes or embayments, which are vertically thermally stratified, and for flowing streams, which exhibit longitudinally varying temperature. One-dimensional models are typically used for these situations. An example is the one-dimensional temperature model in the QUAL-II stream water quality program (Roesner, Giguere and Evenson, 1977a and 1977b).

This paper focuses on one important aspect of water temperature modeling--the computation of surface heat transfer. Emphasis is placed upon this particular topic because it is fairly complicated and thus prone to errors in application. Further, the surface heat flux calculation has extensive and complex data requirements. The means to fill these requirements are not well documented in the literature, although there are many helpful "tricks of the trade" that can be used to assemble the needed data. The purpose of this paper is to describe various techniques to develop input data.

Measurement Units

The measurement units in surface heat transfer calculations have long been a problem. In this paper, I will use English system units since those remain the most used. For heat flux, the English system units are Btu/ft²/day. In the metric system, the preferred units are watt/m² (1 watt = 1 joule/sec). Nevertheless, the Langley (abbreviated Ly), equal to 1 cal/cm², persists in usage. The following conversions are useful:

$$\begin{aligned} 1 \text{ Btu/ft}^2/\text{day} &= 0.131 \text{ watt/m}^2 &= 0.271 \text{ Ly/day} \\ 1 \text{ watt/m}^2 &= 7.61 \text{ Btu/ft}^2/\text{day} &= 2.07 \text{ Ly/day} \\ 1 \text{ Ly/day} &= 0.484 \text{ watt/m}^2 &= 3.69 \text{ Btu/ft}^2/\text{day} \end{aligned}$$

SURFACE HEAT TRANSFER COMPUTATION

The techniques and algorithms to model the transfer of heat through the surface of a water body are well established. Probably the most comprehensive reference on surface heat transfer is a 1972 report by Walter O. Wunderlich of the Tennessee Valley Authority (TVA, 1972). Other excellent references are Ryan and Harleman (1973) and Edinger, Brady and Geyer (1974). The QUAL-II computer program documentation (Roesner, Giguere and Evenson, 1977a and 1977b) is also a good source of information, although it references few original sources.

Surface heat flux consists of five components as illustrated in Figure 1. The water is heated by incoming solar (short-wave) radiation and by incoming atmospheric (long-wave) radiation. Solar radiation is the radiation of the sun, less those portions absorbed by clouds, dust, water vapor and other material in the atmosphere, and less the portion reflected by the water surface. Atmospheric radiation is the radiation emitted by clouds and other material in the atmosphere, less reflection at the water surface. The water body cools by emitting long-wave back radiation and by evaporation and heat conduction. Back radiation emission depends upon the temperature of the water body. Evaporation is basically a diffusion process, driven by the gradient of water vapor pressure from the water surface to the overlying air. Conduction is similar, but driven by the gradient in temperature. Typical magnitudes of the five heat flux components are shown in Table 1.

The sum of the five radiation and heat flux terms is the net heat transfer across the water surface. This is a function of the temperature of the water surface and thus it enters into the equation for water temperature. There are two basic methods to compute the net heat transfer: the complete heat budget and the linearized heat exchange method. Each is discussed in turn in the following.

Heat Budget Method

The complete heat budget requires the separate calculation of the individual heat flux components to arrive at the net surface heat flux. The net heat flux is given as:

$$\Phi = \Phi_{sn} + \Phi_{an} - \Phi_{br} - \Phi_e - \Phi_c \quad (2)$$

where

Φ_n	is the net heat flux into the water surface,
Φ_{sn}	is the net solar (short-wave) radiation into the water surface,
Φ_{an}	is the net atmospheric (long-wave) radiation into the water surface,
Φ_{br}	is the back (long-wave) radiation from the water surface,
Φ_e	is the evaporative heat flux from the water surface, and
Φ_c	is the conductive heat flux from the water surface.

All heat flux components have the English system units of Btu/ft²/day. The formulae for the heat flux components are fairly involved and thus tedious for hand calculations. Nevertheless, they are straightforwardly and quickly computed in digital computer programs.

There is a general consensus within the literature on the appropriate formulae to be employed in computing the individual heat budget terms. The following is a brief presentation of the commonly used formulae drawn largely from Ryan and Harleman (1973).

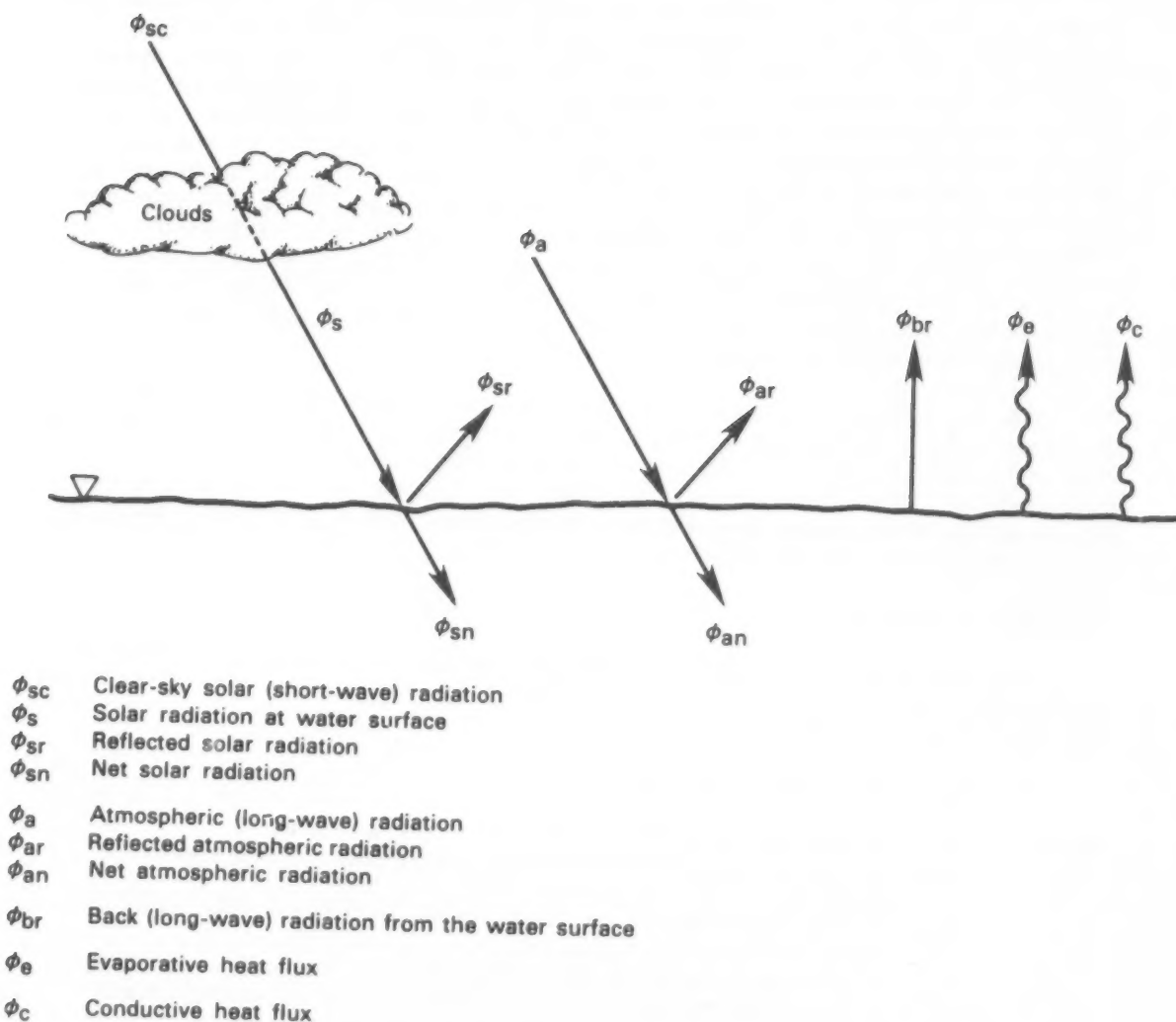


Figure 1 Components of Surface Heat Transfer

Solar Radiation

The net solar radiation into the water surface is the incoming radiation from the sun, less that absorbed or scattered in the atmosphere, blocked by clouds and reflected at the water surface. The best solar radiation information is from measurements at the site, however these are usually unavailable. Lacking measurements, calculations can be made of the solar radiation and its various attenuation mechanisms, thus computing the radiation at the water surface. The report by Wunderlich (TVA, 1972) gives a detailed explanation of the calculation procedures. Unfortunately these procedures are very complex, even for use in computer programs.

A less complicated alternative is recommended by Ryan and Harleman (1973). They suggest that the clear sky solar radiation be determined from empirical information. The clear sky solar radiation is that reaching the water surface during cloudless conditions. It includes the attenuating effects of atmospheric scattering and absorption, but does not include the effects of cloud cover. The net solar radiation is then computed by accounting for reflectance and cloud cover:

$$\phi_{sn} = 0.94 \phi_{sc} (1 - 0.65 C^2) \quad (3)$$

where ϕ_{sc} is the clear sky solar radiation (Btu/ft²/day), and
C is the fraction of the sky covered by clouds.

The factor 0.94 accounts for average reflectance at the water surface following the recommendation of Ryan and Harleman (1973). Determination of ϕ_{sc} is discussed subsequently in this paper.

Equation 3 is an approximation in that it assumes average reflectance and employs clear sky solar radiation. The latter is usually an estimate based on average atmospheric attenuation. Equation 3 works well in nearly all circumstances. However, in certain circumstances attenuating mechanisms are much greater than normal. For example, Locher (1981) reported studies in the Pacific Northwest showing significant atmospheric attenuation due to haze during apparently cloudless conditions. For situations such as these, the more complicated formulae described by Wunderlich (TVA, 1972) are required.

Atmospheric Radiation

Water vapor, carbon dioxide, ozone and other atmospheric constituents cause the atmosphere to radiate as an imperfect black body, or gray body. A perfect black body absorbs all incoming radiation and reemits a radiative flux proportional to the fourth power of its absolute temperature. The constant of proportionality is the Stefan-Boltzmann constant. Gray bodies radiate a fraction of the black body radiation, with the fraction being the emissivity. The emissivity of the atmosphere varies with the air temperature, moisture content and other atmospheric variables. Several formulae for atmospheric radiation were recently evaluated against field data by Hatfield et al. (1983). Although past researchers have tended to recommend the formula by Swinbank (1963), Hatfield's study found the

Swinbank relation to perform poorly. Based on Hatfield's findings, the formula by Brunt (1932) is adequate:

$$\phi_{an} = 2.05 \times 10^{-8} (1 + 0.149 \sqrt{e_a}) (T_a + 460)^4 (1 + 0.17 C^2) \quad (4)$$

where e_a is the vapor pressure 2 meters above the water surface (mm Hg), and
 T_a is the air temperature 2 meters above the water surface ($^{\circ}\text{F}$).

Back Radiation

The water surface, with a nearly constant emissivity of 0.97, approximates a black body. The back radiation from the water surface is given as

$$\phi_{br} = 4 \times 10^{-8} (T_s + 460)^4 \quad (5)$$

where T_s is the water surface temperature ($^{\circ}\text{F}$).

Evaporation

Of the various components of the heat budget, evaporation is the most uncertain. Evaporative heat flux is directly proportional to the rate of evaporative water loss:

$$\phi_e = \rho L_v E \quad (6)$$

where E is the evaporation rate (ft/day),
 L_v is the latent heat of evaporation (Btu/lb), and
 ρ is the density of water (lb/ft³).

TABLE 1
 TYPICAL MAGNITUDES OF SURFACE HEAT FLUX COMPONENTS

	<u>Btu/ft²/day</u>	<u>Watt/m²</u>
Solar radiation, ϕ_{sn}	400 to 2500	50 to 350
Atmospheric radiation, ϕ_{an}	1500 to 3000	200 to 400
Back radiation, ϕ_{br}	2000 to 4000	250 to 500
Evaporation, ϕ_e	0 to 2500	0 to 350
Conduction, ϕ_c	-500 to 1500	-70 to 200

The latent heat of evaporation in Btu/lb is given as a function of temperature as

$$L_v = 1087 - 0.54 T_s \quad (7)$$

Some researchers take L_v as a constant corresponding to $T_s = 212^\circ\text{F}$, the boiling point of water. This is incorrect since T_s will be much less in environmental situations.

There is an extensive literature addressing the calculation of evaporation for both natural and artificially heated water bodies. The general form of the equation for evaporation, E , is:

$$E = F(W) (e_s - e_a) \quad (8)$$

where E is the evaporation rate (ft/day),
 e_s is the saturation vapor pressure of the air at the temperature of the water surface (mm Hg),
 e_a is the vapor pressure at 2 meters above the water surface (mm Hg),
 $F(W)$ is the wind speed function (ft/day/mm hg), and
 W is the wind speed 2 meters above the water surface (miles/hr).

Usually, Equations 6 and 8 are combined and a constant value of L_v assumed to define ϕ_e as

$$\phi_e = f(W) (e_s - e_a) \quad (9)$$

where $f(W)$ is the heat flux wind speed function (Btu/ft²/day/mm Hg):

$$f(W) = \rho L_v F(W)$$

A value of L_v corresponding to a surface temperature of approximately 50 to 70 °F is typically assumed. I will follow that procedure in this paper and assume $L_v = 1060$ Btu/lb as do Ryan and Harleman (1973) and Helfrich et al. (1982). In this way, the values of coefficients in $f(W)$ shown in this paper will be consistent with the literature. Nevertheless, there is little computational effort in computing L_v as a function of T_s . Thus, I recommend a correction factor for computing ϕ_e :

$$\phi_e = \frac{1087 - 0.54 T_s}{1060} f(W) (e_s - e_a) \quad (10)$$

or

$$\phi_e = (1.03 - 5.1 \times 10^{-4} T_s) f(W) (e_s - e_a) \quad (11)$$

Many different expressions have been advanced for the wind speed function $f(W)$. Helfrich et al. (1982) give a systematic review and evaluation of the major formulae. Most expressions adhere to the general form

$$f(W) = a + b W \quad (12)$$

where a is a factor with units Btu/ft²/day/mm Hg, and
 b is a factor with units Btu-hr/ft²/day/mm Hg/mile.

Table 2 shows some of the many formulae available and Figure 2 gives an indication of their variability.

Clearly, temperature model users are faced with a bewildering variety of choices in computing evaporation and the evaporative heat flux. The choice is narrowed by defining whether a natural condition is to be modeled or if the water body is artificially heated by thermal discharges. For natural conditions, the Lake Hefner equation is recommended owing to its thorough calibration and long standing:

$$f(W) = 17 W_2 \quad (13)$$

where W_2 is the wind speed at 2 meters above the water surface (miles/hr).

For artificially heated conditions, the study by Helfrich et al. offers guidance. Helfrich et al. recalibrate various evaporation equations to achieve a best match with field data collected at several cooling ponds. The calibration parameter is a single multiplicative factor that is applied to each formula. The equations giving acceptable results after recalibration are the Meyer, Rimsha and Donchenko, Ryan and Harleman, and Throne equations. Overall, Helfrich et al. recommend the Ryan and Harleman equation owing to its sounder theoretical basis. The recalibrated Ryan and Harleman equation is:

$$f(W) = 17.7 \Delta\theta_v^{1/3} + 11.1 W_2 \quad (14)$$

where $\Delta\theta_v$ is the difference between the virtual temperature at the water surface and in the air 2 meters above the water surface (°F).

The virtual temperature is used in Equation 14 to account for the buoyancy of the moist air above the heated water surface. The virtual temperature is the temperature of dry air with the same density as the moist. It is defined by Ryan and Harleman (1973) as:

$$\theta_v = \frac{T + 460}{1 + .378 e/p} - 460 \quad (15)$$

TABLE 2
EVAPORATION EQUATIONS

<u>Reference</u>	<u>Equation for $f(W)$</u>	<u>Conditions</u>
Lake Hefner equation Harbeck (1952)	$17 W_2$	Natural Lakes
Meyer (1942)	$80 + 10 W_2$	Natural Lakes and Ponds
USGS/Chattahoochee River Barnwell (1982)	$60 + 10.2 W_2$	Natural River
QUAL-II Roesner et al. (1977)	$42.5 + 16.9 W_2$	Rivers
Brady, Graves and Geyer (1969)	$70 + W_2^2$	Cooling Ponds
Ryan and Harleman (1973)	$22.4 (\Delta\theta_v)^{1/3} + 14 W_2$	Cooling Ponds
Rimsha and Donchenko (1957)	$61 + 1.47 (T_g - T_a) + 13.3 W_2$	Heated Streams in Winter
Throne (1951)	$67 + 71 W_2$	Cooling Pond

Note: these equations do not include recalibration factors computed by Helfrich et al. (1982).

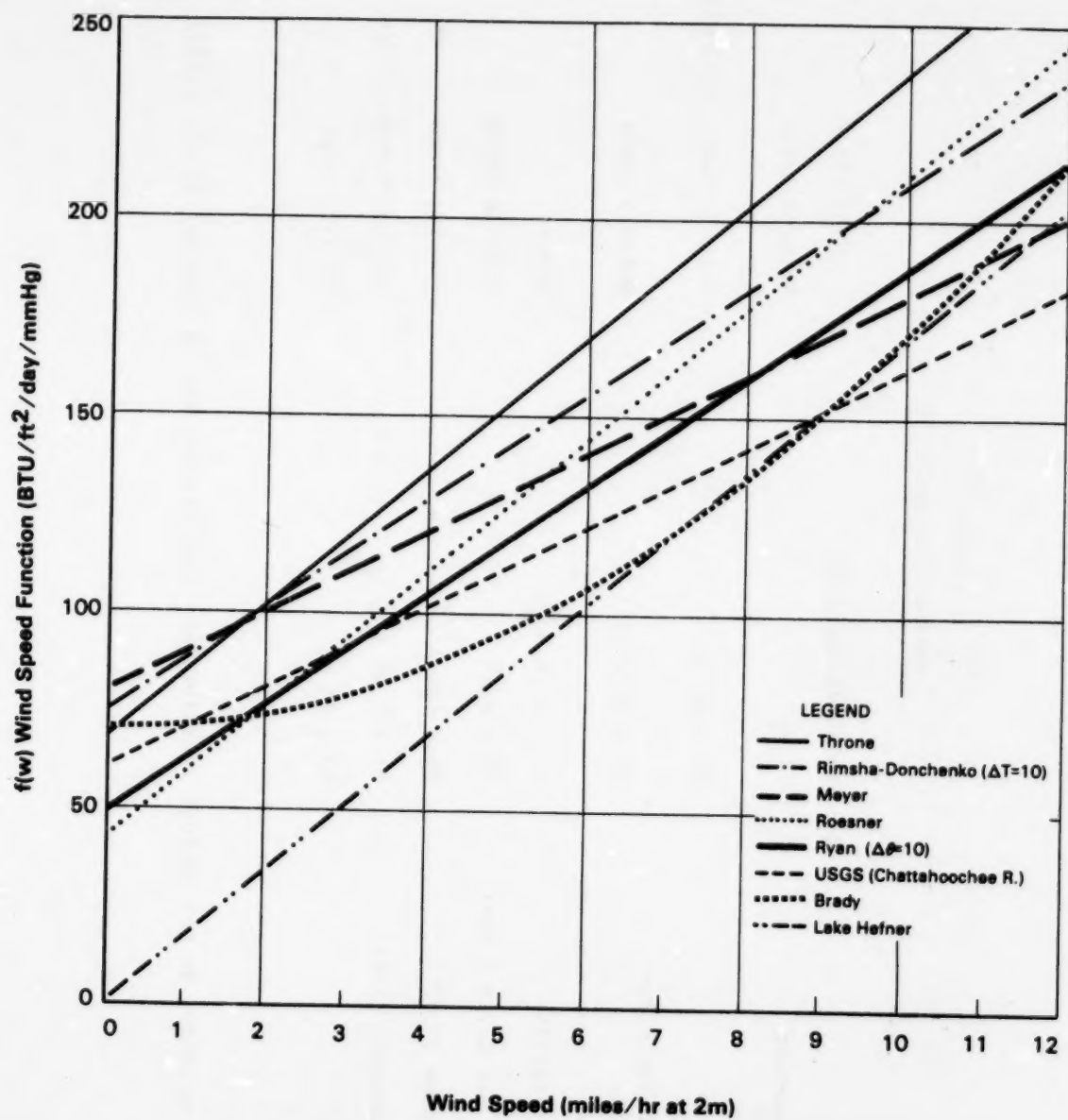


Figure 2 Various Evaporation Wind Speed Functions vs Wind Speed

where θ_v is the virtual temperature ($^{\circ}\text{F}$),
 T is the air temperature ($^{\circ}\text{F}$),
 e is the air vapor pressure (mm Hg), and
 p is atmospheric pressure (mm Hg).

Many computer programs will not accomodate Equation 14 without modification of the program code. For these, the Meyer equation as recalibrated by Helfrich et al. is recommended for artificially heated waters:

$$f(W) = 68 + 8.5 W_2 \quad (16)$$

Alternatively, the wind speed function could be determined by calibration to site data in those applications where sufficient data exist. The formulae of Table 2 should guide such a calibration effort.

In summary, the evaporative heat flux is given as:

$$\phi_e = (1.03 - 5.1 \times 10^{-4} T_s) f(W) (e_s - e_a) \quad (17)$$

where $f(W)$ is selected according to whether the water body is natural or artificially heated.

Conduction

Conduction occurs by a heat diffusion process similar to the moisture diffusion that drives evaporation. Thus, the equation for conductive heat flux is similar in form to that for evaporative heat flux:

$$\phi_c = 0.255 f(W) (T_s - T_a) \quad (18)$$

Net Heat Flux

As stated previously, the net heat flux is the sum of the five component fluxes outlined above. English and metric system equations for the components are summarized in Table 3. The net heat flux is a function of the surface water temperature, the overlying air temperature, moisture content and wind speed, the incoming solar radiation, and the cloud cover and condition of the atmosphere.

Linearized Heat Exchange Method

The linearized heat exchange concept was popularized by John Edinger and others from The Johns Hopkins University. In this approach, the net heat flux is assumed to be a linear function of the surface water temperature:

$$\phi_n = K (T_s - T_E) \quad (19)$$

where K is the surface heat exchange coefficient ($\text{Btu}/\text{ft}^2/\text{day}/^{\circ}\text{F}$), and
 T_E is the equilibrium temperature ($^{\circ}\text{F}$).

TABLE 3
SUMMARY OF EQUATIONS FOR SURFACE HEAT FLUX COMPONENTS

Component	English System	Metric System (S.I.)
Heat Flux Components:		
Net Solar Radiation	ϕ in Btu/ft ² /day	ϕ in watt m ²
Net Atmospheric Radiation	$\phi_{sn} = 0.94 \phi_{sc} (1-0.65 c^2)$	$\phi_{sn} = 0.94 \phi_{sc} (1-0.65 c^2)$
Back Radiation	$\phi_{an} = 2.05 \times 10^{-8} (1 + 0.149 \sqrt{e_a}) (T_a + 460)^4 (1+0.17 c^2)$	$\phi_{an} = 2.89 \times 10^{-8} (1 + 1.29 \sqrt{e_a}) (T_a + 273)^4 (1+0.17 c^2)$
Evaporation	$\phi_{br} = 4.0 \times 10^{-8} (T_a + 460)^4$	$\phi_{br} = 5.44 \times 10^{-8} (T_a + 273)^4$
Conduction	$\phi_e = (1.03 - 5.1 \times 10^{-4} T_a) f(W) (e_a - e_s)$	$\phi_e = (1.01 - 9.1 \times 10^{-4} T_a) f(W) (e_a - e_s)$
	$\phi_c = 0.255 f(W) (T_a - T_s)$	$\phi_c = 0.60 f(W) (T_a - T_s)$
Wind Speed Functions:		
Natural Conditions	$f(W)$ in Btu/ft ² /day/mm Hg	$f(W)$ in watt/m ² /mb
Artificially heated conditions (Ryan and Harleman 1973)	$f(W) = 17 W_2$	$f(W) = 3.84 W_2$
Artificially heated conditions (Meyer 1942)	$f(W) = 17.7 \Delta \theta_v^{1/3} + 11.1 W_2$	$f(W) = 2.18 \Delta \theta_v^{1/3} + 2.51 W_2$
	$f(W) = 68 + 8.5 W_2$	$f(W) = 6.87 + 1.92 W_2$
Parameters:		
C, cloud cover	fraction of sky	fraction of sky
e_a , air vapor pressure at 2 m above water	mm mercury (mm Hg)	millibar (mb)
e_s , saturation vapor pressure at temperature T_s	mm mercury	millibar
T_a , air temperature	degrees F	degrees C
T_s , water surface temperature	degrees F	degrees C
W_2 , wind speed at 2 m above water	miles/hour	m/s
$\Delta \theta_v$, virtual temperature difference between water surface and 2 m (see text)	degrees F	degrees C

* Wind speed functions recalibrated by Helfrich et al. (1982)

Equation 19 entails two parameters that are artificial, but nevertheless useful and intuitive. The equilibrium temperature is the water surface temperature that is in equilibrium with the environment: the net surface heat transfer is zero. Water bodies continually seek their equilibrium temperature, losing or gaining heat as required. But since environmental conditions change continuously, the equilibrium temperature also changes and a water body is seldom at its equilibrium temperature.

The surface heat exchange coefficient is the incremental change in the rate of net heat transfer per unit change in surface water temperature. It varies with the surface temperature and thus should be recalculated as the water temperature changes.

The linearized heat exchange method is attractive since it permits analytical solutions for a variety of water temperature problems. However, it is less accurate than the heat budget method and not suitable for detailed design or analysis studies. A general recommendation is that the linearized method not be used in computer modeling since there is no difficulty or additional data required to use the more accurate heat budget method.

Despite the recommendation that the linearized method not be used, the equilibrium temperature concept is very useful and attractive. It is particularly helpful in screening data--for example in reviewing historical records to select a critical period for design. Extended periods of extremely high equilibrium temperature are those in which thermal aquatic impacts will be greatest. These same periods are when cooling ponds and similar heat rejection systems perform their worst. Thus, the equilibrium temperature is a useful variable in selecting simulation periods from long data records. It is also valuable to understanding water temperature trends in transient computer simulations. Table 4 is a BASIC-language computer program for calculation of the equilibrium temperature.

METEOROLOGICAL DATA REQUIREMENTS

Probably the greatest burden in water temperature modeling is the extensive input data requirement. The calculation of the heat budget requires meteorological data characterizing the air temperature, humidity, wind speed, cloudiness, and solar radiation at the study site. It is rare that a complete set of data is available from measurements at a site, and thus other sources must be sought.

The most comprehensive source of meteorological data is the published observations of the National Climatic Center in Asheville, NC. The center publishes data for many locations throughout the country in a number of formats. Monthly summaries are published for each state in the publication Climatological Data and daily and 3-hourly data are published each month for selected weather stations in Local Climatological Data. Data are also available on magnetic tape for computer modeling of long periods.

TABLE 4
BASIC-LANGUAGE COMPUTER PROGRAM FOR CALCULATION
OF EQUILIBRIUM TEMPERATURE

```

100 REM =====
110 REM CALCULATION OF
120 REM EQUILIBRIUM TEMPERATURE
130 REM =====
140 DIM U(3),H(3)
150 DISP "INPUT T, W, R, S, S2, C, F."
160 DISP "ENTER STOP TO END RUN."
170 DISP " "
180 DISP "T IS THE AIR TEMPERATURE, DEG F."
190 DISP "W IS THE WIND SPEED, MILES PER HOUR AT 2 M."
200 DISP "R IS THE RELATIVE HUMIDITY, PERCENT."
210 DISP "S IS THE CLEAR SKY SOLAR RADIATION, BTU/SQ FT/DAY."
220 DISP "S2 IS THE OBSERVED SOLAR RADIATION, BTU/SQ FT/DAY."
230 DISP "C IS THE CLOUD COVER, FRACTION."
240 DISP "F IS THE WIND SPEED FUNCTION."
250 DISP "F=1 FOR LAKE HEFNER."
260 DISP "F=2 FOR MODIFIED RYAN."
270 DISP "F=3 FOR MODIFIED MEYER."
280 DISP "F=4 FOR USER SPECIFIED."
290 DISP " "
300 INPUT T,W,R,S,S2,C,F
310 IF F=1 OR F=2 OR F=3 THEN 340
320 DISP "ENTER A/B FOR WIND FUNCTION F(W)=A/BW."
330 INPUT A,B
340 R=RW/100
350 T=1+460
360 IF S2>0 THEN 390
370 REM ---COMPUTE SOLAR RADIATION FROM CLEAR SKY RADIATION---
380 S2=S*(1-.65*CW)
390 H3=.94852
400 REM ---COMPUTE AIR VAPOR PRES AND VIRTUAL TEMP---
410 E1=FNE(T)*R
420 T3=T/(1-.378*E1/760)-460
430 REM ---COMPUTE LONG WAVE RADIATION (ATMOSPHERIC)---
440 H2=.000000021*(1+.149*SGR(E1))*T4-48*(1+.17*CW)
450 REM USE BISECTION METHOD TO COMPUTE EQUILIBRIUM TEMP
460 REM -----
470 I1=1
480 I2=3
490 U(1)=T-5
500 U(2)=T
510 U(3)=T+5
520 Z1=0
530 FOR I=11 TO 12
540 Z1=Z1+1
550 U1=U(1)+460
560 REM ---COMPUTE LONG WAVE BACK RADIATION---
570 H1=.00000004*U1^4
580 E2=FNE(U(1))
590 IF F01 THEN 630
600 REM LAKE HEFNER WIND FUNCTION
610 F1=17*W
620 GOTO 770
630 IF F#2 THEN 710
640 REM RYAN WIND FUNCTION
650 D2=U1/(1-.378*E2/760)-460
660 D2=D2-13
670 D3=.0024*W*W*W
680 IF D2<D3 THEN D2=D3
690 F1=17.7*D2*.3334+11.1*W
700 GOTO 770
710 IF F#3 THEN 750
720 REM MEYER WIND FUNCTION
730 F1=.6848*.5*W
740 GOTO 770
750 REM SPECIFY A/BW WIND FUNCTION
760 F1=A/BW
770 REM ---COMPUTE EVAP HEAT FLUX---
780 H4=F1*(E2-E1)
790 REM ---COMPUTE CONDUCTION---
800 H5=H4*.255*(U(1)-T)/(E2-E1)
810 IF H4<0 THEN H4=0
820 H(I)=H3+H2-H1-H4-H5
830 REM DETERMINE IS H HAS CONVERGED TO 1 BTU/SQ FT/DAY OR LESS
840 REM -----
850 IF ABS(H(I))<1 THEN 1030
860 NEXT I
870 I1=2
880 I2=2
890 IF H(I)<0 THEN 990
900 IF H(I)>0 THEN 960
910 J1=1
920 IF H(I)*H(2)<0 THEN J1=3
930 U(J1)=U(2)
940 H(J1)=H(2)
950 GOTO 1010
960 U(3)=U(3)+10
970 I2=3
980 GOTO 1010
990 U(1)=U(1)-10
1000 I1=1
1010 U(2)=(U(1)+U(3))/2
1020 GOTO 530
1030 REM EQUILIBRIUM TEMPERATURE FOUND
1040 REM -----
1050 DISP "EQUILIBRIUM TEMPERATURE, DEG F = ",U(2)
1060 GOTO 290
1070 REM =====
1080 REM SATURATION VAPOR PRESSURE FUNCTION
1090 REM =====
1100 DEF FNE(T) = 25.4*EXP(17.62-9500.8/(T+460))
1110 END

```

Some other, less comprehensive sources of data are useful for rough calculations. These are the "Climatic Atlas of the United States", published by the National Oceanic and Atmospheric Administration (1977) and an EPA report, "Effect of Geographical Variation on the Performance of Recirculating Cooling Ponds" (Thackston, 1974). The EPA publication also includes extreme monthly conditions which are useful to evaluate surface heat transfer when it is at its least.

The representativeness of off-site data is rarely addressed. Nevertheless, it is an important matter. The weather bureau station nearest the site need not be the most representative. It may fall in a different geographic or climatic province, and thus be inappropriate. A rigorous attempt to synthesize on-site data was made by Jirka et al. (1977) for the site of the North Anna Power Station in Virginia. They used statistical techniques to correlate a single year of on-site meteorological data with coincident records at three surrounding weather stations. Then, they employed their computed statistical correlations to generate a long-term synthetic record for the site from the records of the weather stations. A similar procedure is recommended for detailed design studies where the representativeness of off-site data is uncertain. In any study, the modeler should consider the local climatology to determine the most representative station for off-site meteorological data.

Having obtained data, it is often necessary to convert it to a format compatible with the heat budget formulae. Air temperature presents no problem, but humidity, wind speed, cloudiness and solar radiation often need adjustments or conversions.

Humidity Data

The humidity of air may be expressed directly as relative humidity, and indirectly through the dew point or wet bulb temperature. The wet bulb or dew point, together with the air (or dry bulb) temperature, may be used to compute relative humidity using a psychrometric chart. The chart, with instructions, is given in the Handbook of the American Society of Heating, Refrigerating and Air-Conditioning Engineers (ASHRAE, 1981) and numerous other publications. Another possible conversion method from wet bulb to relative humidity is available by inverting an empirical equation given by Thackston (1974):

$$T_{wb} = (0.655 + 0.36 R_H) T_a \quad (20a)$$

or

$$R_H = 2.78 \frac{T_{wb}}{T_a} - 1.82 \quad (20b)$$

where T_{wb} is the wet bulb temperature ($^{\circ}F$), and
 R_H is the relative humidity (expressed as a fraction).

This equation is valid for relative humidity less than about 95%. TVA (1972) gives the following conversion from dew point temperature to air vapor pressure over water:

$$e_a = \exp [2.3026 \left(\frac{7.5 T_d - 236.9}{T_d + 395.5} + 0.6609 \right)] \quad (21)$$

where T_d is the dew point temperature of the air ($^{\circ}F$).

The humidity conditions of the air must be expressed as the water vapor pressure in the equations for calculation of evaporative and conductive heat flux. The vapor pressure of the air is computed as:

$$e_a = R_H e_{sat}$$

where e_{sat} is the saturation vapor pressure (mm Hg).

Calculation of surface heat transfer requires that the saturation vapor pressure be determined as a function of temperature. Extremely accurate and precise tables of this function are found in the Smithsonian Meteorological Tables (List, 1971). Several different empirical formulae have been proposed to compute the saturation vapor pressure as a function of temperature. For environmental temperatures, the empirical equation by Thackston (1974) is virtually indistinguishable from the tabulated function:

$$e_{sat} = 25.4 \exp \left(17.62 - \frac{9500.8}{T + 460} \right) \quad (22)$$

where T is the temperature ($^{\circ}F$).

In computing evaporative heat flux, the vapor pressure at the water surface is computed by assuming it is saturated ($R_H = 1.0$) and at the temperature of the water. The vapor pressure at a height above the surface is based upon ambient temperature and relative humidity of the air.

Wind Speed

The height above the water or ground surface at which the wind is measured is an important consideration in the use of wind speed data. Often, the measuring height is unknown. But if it is available, the speed may be adjusted to the height presumed by the heat transfer formulae (usually 2 meters). Based upon an assumed logarithmic wind speed profile, Ryan and Harleman (1973) give the following adjustment relation:

$$\frac{W_z}{W_{z_1}} = \frac{\ln \left(\frac{z}{z_0} \right)}{\ln \left(\frac{z_1}{z_0} \right)} \quad (23)$$

where W_z is the (desired) wind speed at height z ,
 W_{z_1} is the (known) wind speed at height z_1
 z is the height above the water surface presumed by the surface heat transfer formula (m),
 z_1 is the height above the water at which the wind speed is known (m), and
 z_0 is the wind roughness height (m).

Helfrich et al. (1982) discuss the selection of z_0 . A value of $z_0 = 0.001$ meter is a good approximation for most wind conditions. At high wind speeds, waves form on the water surface and increase z_0 .

Cloudiness

The cloud cover of the sky is usually recorded as the percent (or number of tenths) of the sky that is covered by clouds. Occasionally, the quantity percent possible sunshine will be given. The conversion from percent possible sunshine to cloud cover can be made using an equation derived from relations given by TVA (1972):

$$C = [1.2 (1-P^{2/3})]^{1/2} \quad (24)$$

where C is the cloud cover (expressed as a fraction) and
 P is the possible sunshine (expressed as a fraction).

Solar Radiation

Direct measurements of solar radiation are rare, and consequently these data must be determined by other means. Neglecting the influence of the clouds, the clear sky solar radiation may be determined as a function of the geographical latitude, the time of year, and the hour of the day. TVA (1972), Brock (1981) and any of a variety of references from the meteorological and heating and air conditioning engineering literature give formulae to calculate the clear sky solar radiation. Unfortunately, these formulae are complex and cumbersome, even for computer calculations. The complexity derives from the need for esoteric parameter values and the fact that clock time cannot be used in the calculations, only solar time (i.e., relative to true solar noon) can be used.

If the modeler desires only daily average values of solar radiation, there are simple alternatives. For example, Thackston (1974) employs curve-fit equations that are reasonably accurate and amenable to both hand and computer calculations. Table 5 is a BASIC-language computer program

TABLE 5
BASIC-LANGUAGE COMPUTER PROGRAM FOR CALCULATION
OF DAILY AVERAGE CLEAR SKY SOLAR RADIATION

```

100 REM =====
110 REM CALCULATION OF CLEAR SKY
120 REM SOLAR RADIATION AT WATER
130 REM IN UNITS BTU/SQ FT/DAY
140 REM =====
150 REM BASED ON THACKSTON, 1974
160 REM REPORT EPA-660/2-74-085
170 REM
180 REM INITIALIZE DATA
190 REM -----
200 REM DAY OF YEAR ARRAY
210 DIM D9(12)
220 FOR I=1 TO 12
230 READ D9(I)
240 NEXT I
250 DATA 0,31,59,90,120,151,181,212,243,273,304,334
260 REM COEFFICIENTS FOR CURVE FIT EQUATIONS
270 DIM C1(21),C2(21),C3(21)
280 FOR I=1 TO 21
290 READ C1(I),C2(I),C3(I)
300 NEXT I
310 REM INPUT LATITUDE AND DATE
320 REM -----
330 DISP "ENTER SITE LATITUDE "
340 DISP " (26 TO 46 DEGREES) "
350 DISP " (0 TO END RUN) "
360 INPUT L
370 IF L=0 THEN STOP
380 L1=INT(L)
390 L3=L1-25
400 L2=CEIL(L)
410 L4=L2-25
420 IF L1<26 OR L1>46 THEN 330
430 IF L2<26 OR L2>46 THEN 330
440 DISP "ENTER DATE AS MONTH, DAY"
450 INPUT D7,D8
460 D=D9(D7)+D8
470 REM COMPUTE CLEAR SKY SOLAR
480 REM -----
490 D6=283.14159/D/366
500 S=C1(L3)-C2(L3)*SIN(C3(L3)+D6)
510 IF L1=L2 THEN 560
=====
520 S2=C1(L4)-C2(L4)*SIN(C3(L4)+D6)
530 L1=L-L1
540 L2=L2-L
550 S=L2*S+L1*S2
560 S=24*S
570 REM DISPLAY ANSWER
580 REM -----
590 DISP USING 600 ; L
600 IMAGE "AT LATITUDE ",DD,D," DEGREES"
610 DISP USING 620 ; D7,D8
620 IMAGE "ON DATE ",DD,"/",DD
630 DISP USING 640 ; S
640 IMAGE "CLEAR SKY SOLAR RADIATION IS /* ",SDD," BTU/SQ FT/DAY"
650 REM RERUN
660 REM -----
670 GOTO 330
=====
680 REM CURVE FIT EQUATION
690 REM PARAMETERS
700 REM =====
710 REM -----
720 DATA 80.155,29.207,1.679
730 DATA 79.371,30.236,1.713
740 DATA 78.566,31.219,1.71
750 DATA 77.604,32.145,1.74
760 DATA 76.655,33.156,1.728
770 DATA 76.041,34.133,1.694
780 DATA 75.06,35.194,1.737
790 DATA 74.046,35.938,1.734
800 DATA 73.161,36.834,1.727
810 DATA 72.248,37.699,1.738
820 DATA 71.35,38.598,1.721
830 DATA 70.394,39.413,1.73
840 DATA 69.35,40.188,1.741
850 DATA 68.362,40.982,1.739
860 DATA 67.281,41.706,1.742
870 DATA 66.24,42.442,1.736
880 DATA 65.197,43.128,1.74
890 DATA 64.113,43.788,1.739
900 DATA 63.014,44.471,1.739
910 DATA 61.911,45.021,1.74
920 DATA 60.782,45.639,1.735
930 END

```


utilizing his equations. Another very workable alternative for hand calculation is a graphical look-up, as presented by Hamon, Weiss and Wilson (1954). Figure 3 is a graph of clear sky solar radiation based on their empirical results.

As stated above, distribution of solar radiation throughout the day can be difficult. If one only desires to construct a representative daily variation in solar radiation, then the formulae of TVA (1972) or Brock (1981) are quite useful. However, if one wishes to construct a diurnal variation to be used with measured data, then the differences between clock time and solar time require careful attention. For example, I have attempted in the past to back-calculate cloud cover from the differences between measured solar radiation at a site (ϕ_g in Figure 1) and a computed value of clear sky solar radiation (ϕ_{cs}). The procedure worked fairly well for daily averages (Wells et al., 1982). But it worked very poorly for hourly values due to inexact coordination of clock time and solar time.

For some problems, a hand calculation procedure for diurnal variation of solar radiation is desirable. For these, it is possible to distribute daily solar radiation throughout the day using data given in Chapter 27 of the ASHRAE Handbook, 1981 Fundamentals. In the handbook, tables of solar position and intensity for various latitudes give the direct normal irradiation distribution by hours during the day. The normalization of the hourly values by the sum for the day supplies a set of distribution constants. These distribution constants may be applied to the daily average clear sky solar radiation to construct the diurnal variation of solar radiation.

One final, but very important footnote applies to both the solar radiation and atmospheric radiation components. These radiation fluxes can be measured directly to supply far more accurate data than the calculation procedures above.

SUMMARY

This paper is a review of the surface heat transfer calculations necessary to model water temperature in surface water bodies. The paper recommends the use of the heat budget technique as more accurate than the linearized method. The linearized method is based on the equilibrium temperature and surface heat exchange coefficient. Within the heat budget technique, there are the following heat flux components and recommended procedures:

Solar Radiation -

It is best to measure this directly. If measurements are not available, clear sky solar radiation may be estimated from Figure 3 or Table 5 and then modified for cloud cover and reflectance (Equation 3). A more exact procedure is recommended for areas with prevalent haze, dust or high water vapor.

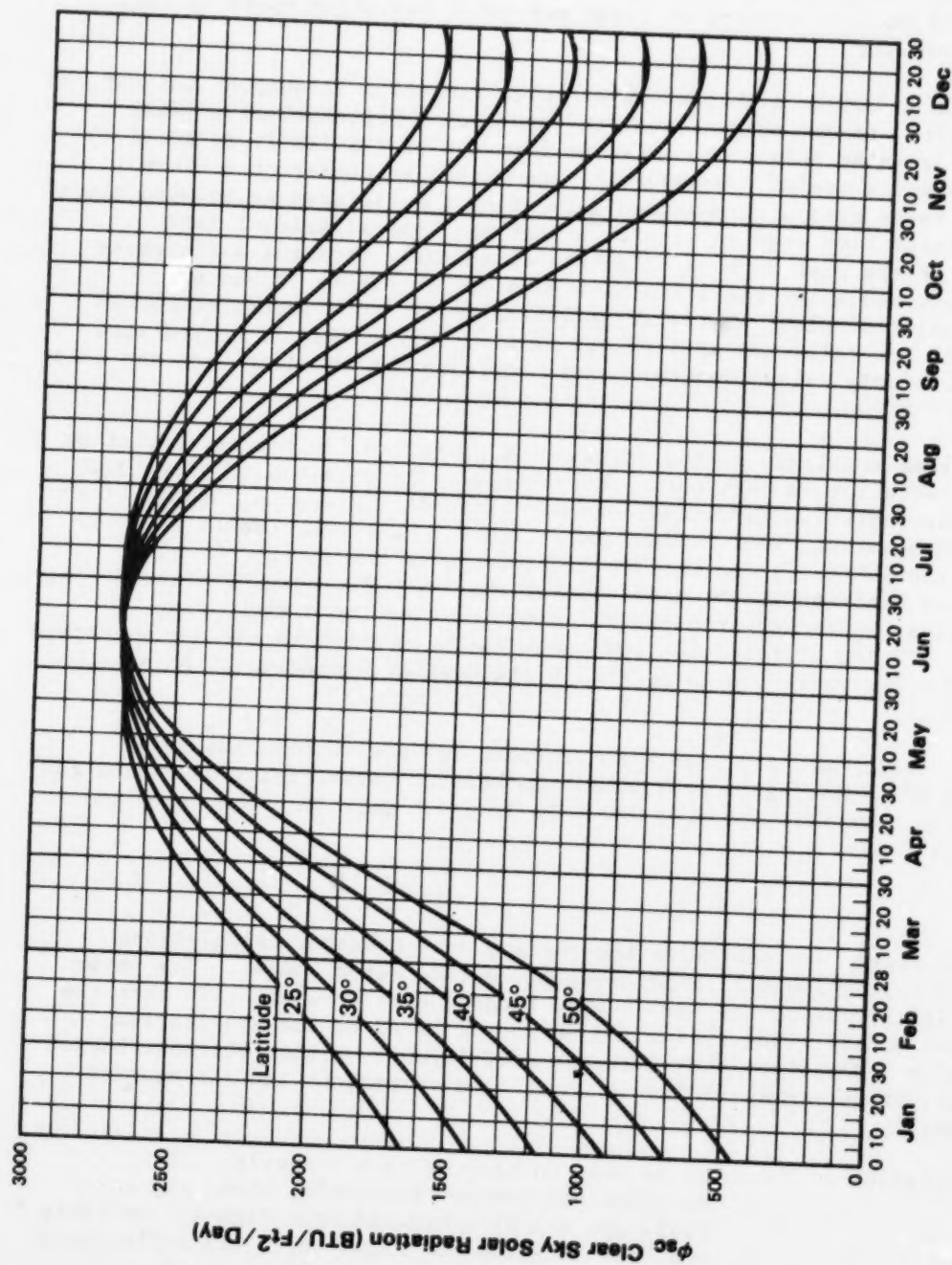


Figure 3 Clear Sky Solar Radiation According to Hamon, Weiss and Wilson (1954)

Atmospheric Radiation - It is best to measure this directly. If measurements are not available, Brunt's equation (Equation 4) is recommended.

Back Radiation - This flux is accurately determined by Equation 5.

Evaporation - Evaporative heat flux is computed as the product of the latent heat of evaporation, the gradient in vapor pressure from the water surface to the overlying air, and an empirical wind speed function. The Lake Hefner wind speed function is recommended for natural water bodies, the Ryan-Harleman for artificially heated. In computer programs that will not accomodate the Ryan-Harleman equation, the Meyer equation is a good substitute. Both the Ryan-Harleman and Meyer equations should include the recalibration factors by Helfrich et al. (1982).

Conduction - Conductive heat flux is the product of a constant of proportionality the gradient in temperature from the water surface to the overlying air, and the wind speed function.

Table 3 is a complete summary of the recommended equations for the heat budget. Equations in both English and metric units are included in Table 3.

The input data required for water temperature modeling are extensive. Five types of data are required: the air temperature, the wind speed, the relative humidity, the cloud cover, and the solar radiation or an estimate of clear sky solar radiation. This paper includes descriptions of procedures to manipulate data as follows:

Humidity - conversion from wet bulb to relative humidity
 conversion from dew point to relative humidity
 calculation of saturation vapor pressure
 calculation of vapor pressure

Wind Speed - adjustment for measurement height

Cloudiness - conversion from percent possible sunshine to cloud cover

Solar Radiation - graphical estimation of clear sky solar radiation
 computer program to calculate estimated clear sky solar radiation
 diurnal distribution of clear sky solar radiation

The paper also includes a computer program to compute the equilibrium temperature as a function of the meteorological variables above.

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Although I hope this paper is a useful collection of information, I freely confess that few of the ideas presented are mine originally. I gratefully acknowledge the instructions and ideas of my former colleagues Patrick J. Ryan, Martin S. Leonard, Fred H. Wend and Frederick A. Locher at Bechtel and Donald R.F. Harleman, E. Eric Adams and Keith D. Stolzenbach at MIT. My thanks to Margaret Barnett, Linda Blacksmith, Rose Rondeau, Michelle King and Janet Mahoney for preparing the text, tables and figures.

REFERENCES

- ASHRAE (American Society of Heating, Refrigerating and Air Conditioning Engineers) 1981. ASHRAE Handbook: 1981 Fundamentals. ASHRAE, Atlanta, GA.
- Barnwell, T. 1982. Personal communication. U.S. Environmental Protection Agency, Center for Water Quality Modeling, Athens, GA.
- Brock, T.D. 1981. Calculating Solar Radiation for Ecological Studies. Ecological Modelling, 14:1/2, pp. 1-19. November 1981.
- Brunt, D. 1932. Notes on Radiation in the Atmosphere. Quarterly Journal of The Royal Meteorological Society, 58, pp. 389-418.
- Edinger, J.E., D.K. Brady and J.C. Geyer 1974. Heat Exchange and Transport in the Environment. Report No. 14, Research Project RP-49. Electric Power Research Institute, Palo Alto, CA.
- Hamon, R.W., L.L. Weiss and W.T. Wilson 1954. Insolation as an Empirical Function of Daily Sunshine Duration. Monthly Weather Review, 82:6, pp. 141-146. June 1954.
- Harbeck, G.E. 1952. The Lake Hefner Water Loss Investigation. Circular 229. U.S. Geological Survey.
- Hatfield, J.L., R.J. Reginato and S.B. Idso 1983. Comparison of Long-Wave Radiation Calculation Methods Over the United States. Water Resources Research, 19:1, pp. 285-288. February 1983.
- Helfrich, K.R., E.E. Adams, A.L. Godbey and D.R.F. Harleman 1982. Evaluation of Models for Predicting Evaporative Water Loss in Cooling Impoundments. Report CS-2325, Research Project 1260-17. Electric Power Research Institute, Palo Alto, CA. March 1982.
- Jirka, G.H., D.N. Brocard, K.A. Hurley Octavio, M. Watanabe and D.R.F. Harleman 1977. Analysis of Cooling Effectiveness and Transient Long-Term Simulations of a Cooling Lake with application to the North Anna Power Station. Report No. 232. Ralph M. Parsons Laboratory, Department of Civil Engineering, Massachusetts Institute of Technology, Cambridge, MA. December 1977.

- List, R.J. 1971. Smithsonian Meteorological Tables. Sixth Revised Edition. Smithsonian Institution Press, Washington, D.C.
- Locher, F.A. 1981. Personal Communication. Bechtel Civil and Minerals Corporation, San Francisco, CA.
- Meyer, A.F. 1942. Evaporation from Lakes and Reservoirs. Minnesota Resources Commission, St. Paul, MN. June 1942.
- National Oceanic and Atmospheric Administration 1977. Climatic Atlas of the United States. National Climatic Center, Asheville, NC.
- Rimsha, V.A. and R.V. Donchenko 1957. The Investigation of Heat Loss from Free Water Surfaces in Wintertime (in Russian). Leningrad Gosudarstvennyi Gidrologicheskii, 64.
- Rosener, L.A., P.R. Giguere and D.E. Evenson 1977a. User's Manual for the Stream Quality Model QUAL-II. United States Environmental Protection Agency, Center for Water Quality Modeling, Athens, GA.
- Rosener, L.A., P.R. Giguere and D.E. Evenson 1977b. Computer Program Documentation for the Stream Quality Model QUAL-II. United States Environmental Protection Agency, Center for Water Quality Modeling, Athens, GA.
- Ryan, P.J. and D.R.F. Harleman 1973. An Analytical and Experimental Study of Transient Cooling Pond Behavior. Report No. 161. Ralph M. Parsons Laboratory, Department of Civil Engineering, Massachusetts Institute of Technology, Cambridge, MA. January 1973.
- Swinbank, W.C. 1963. Long-wave Radiation from Clear Skies. Quarterly Journal of the Royal Meteorological Society, 89, pp. 339-348.
- Thackston, E.L. 1974. Effect of Geographical Location on Performance of Recirculating Cooling Ponds. Report No. 660/2-74-085. U.S. Environmental Protection Agency. November 1974.
- Throne, R.F. 1951. How to Predict Cooling Lake Action. Power, 95, pp. 86-89. September 1951.
- TVA (Tennessee Valley Authority) 1972. Heat and Mass Transfer Between a Water Surface and the Atmosphere. Water Resources Research Laboratory Report No. 14. Tennessee Valley Authority, Division of Water Control Planning, Engineering Laboratory, Norris, TN. April 1972.
- Wells, S.A., E.E. Adams and D.R.F. Harleman 1982. Calibration and Verification of the Cooling Lake Model for North Anna Power Station during the period July 1978-September 1981. Report No. 272. Ralph M. Parsons Laboratory, Department of Civil Engineering, Massachusetts Institute of Technology, Cambridge, MA. March 1982.

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